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had an effect. The location of any cracks and their size should be carefully noted for future reference and comparison. Evidences of rust at cracks can mean possible damage to prestressing steel.

Pretensioned box sections should be checked during the passage of heavy loads to see whether any unit is acting independently of the others. Such independent action would indicate spreading of the girders or failure of the longitudinal key between girders.

On bridges with underpassing traffic the exterior faces and the soffits of all types of prestressed girders should be examined. Spalling, cracking or damage to prestressing steel should be noted.

Inspections of earthquake restrainer mechanisms and for earthquake damage should be conducted as outlined in Article 3.8.3.2.

3.8.3.4 Timber Systems

Examine timber stringers for splitting, cracking, and excessive deflection. Look for crushing and evidence of decay where they bear on the bent caps or abutment seats and at their top edge where the floor is supported. Stringers should be kept clear of dirt accumulations to help prevent decay from starting and to help prevent its acceleration once it has started.

The bridging between the timber stringers should be checked to see that it is tight and functioning properly. Timber connections should be checked for loose or missing fasteners.

In order to evaluate the capacity of existing timber structures, the following information should be recorded:

(a) The beam size, spacing and span length;
(b) The type of beam: rough sawn, dressed, nail-laminated, or glue-laminated;
(c) Horizontal shear capacity is controlled by beam depth. Have beams been cut or notched at the bearing and to what extent?
(d) Age of timber should be estimated;
(e) The moisture content of the timber should be estimated or measured;
(f) The species and grade of the lumber should be identified. Original and repair construction records should be checked for material delivery slips. Where no information is available, the inspector must use judgment based upon local experience, visual appearance, odor, cross grain, etc. Where more exact information is required, obtain a sample for testing by a laboratory.

The age, moisture content, species and grade of timber are used in establishing values for the allowable timber stresses to be used in the load rating computations. Field grading and/or estimates of allowable stresses may be necessary.

3.8.3.5 Floor Systems

Truss and deck girder structures are constructed with a system of stringers, floor beams and, if present, brackets to transmit the live load from the deck to the main load-carrying members (girders or trusses). The transverse floor beams and/or brackets can be Fracture Critical Members depending on the framing used. A U-bolt floor beam connection to the truss may be an example of a fracture critical detail. The bridge record should clearly indicate whether or not the floor system contains FCMs.

Inspect stringers, floor beams and overhang brackets for cracks and losses due to rust. Floor beams and connections located below deck relief joints frequently show severe rust due to leakage through the deck joint. Floor beam overhanging tie plates should be carefully examined for evidence of cracking or section loss.

Stringer systems are usually provided with simple expansion devices such as slotted holes at the floor beam connections. These expansion devices should be checked for freedom of movement, uplift or other evidence that the floor system is not functioning as designed.

The floor beams are frequently subjected to out-of-plane bending due to restraints imposed by stringer, girder and bracing connections. Check for evidence of fatigue cracks adjacent to the various connection points.

On those bridges where the deck does not bear directly on the main longitudinal members, there is a tendency for the deck and main longitudinal members not to respond to dynamic loading in synchronization which can cause twisting and out-of-plane bending in the floor beams. Check for evidence of fatigue cracks adjacent to the floor beam/girder connections.

3.8.3.6 Trusses

The examination of any truss will normally begin with sighting along the roadway rail or curb and along the truss chord members to determine any misalignment either vertical or horizontal. Check alignment of trusses carefully for any sag which may is usually attached to the stringers with plug welds which are not directly observable. Vertical movement of the deck under the passage of live load may indicate weld failure. The fill material of the wearing surface should be examined for cracks or depressions. Open cracks in the wearing surface will allow rust through of the deck elements to occur at an accelerated rate.

Orthotropic steel plate decks consist of a flat steel plate with a series of stiffening web elements. A wearing surface is bonded to the top of the steel plate. On some structures the steel plate is itself a flange element of a box girder section. The inspector should check for debonding of the overlay, rust through or cracks in the steel plate, and for the development of fatigue cracks in the web elements or connecting welds. The connection between the orthotropic plate deck and supporting members should be checked, where visible, and any evidence of live load movement noted.

3.8.4.4 Timber Decks

Timber decks should be examined for decay especially at their contact surfaces where they bear on the stringers and between layers of planking or laminated pieces. Note any looseness which may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic will reveal looseness or excessive deflection in the members.

3.8.4.5 Expansion Joints

Expansion joints provide for thermal expansion of the deck and superstructure. They should be checked for freedom of expansion. The clear opening of the joint should provide for adequate expansion of the adjacent superstructure elements considering the span lengths and temperature at the time of inspection. The inspector should measure expansion joint openings and ambient temperature at easily identifiable locations, so that future inspections can establish a record of joint movement over time. Inspect for solid objects (noncompressibles) which can become wedged in the joint and prevent joint contraction.

On joints without armoring, inspect for proper joint alignment, the presence and condition of any joint sealant material, and for evidence of spalls or "D" cracking in the slab edges which would prevent proper sealing of the joint.

Armored joints, without sealant material, such as sliding plate dams or finger joints should be inspected both above and below deck for the condition of the supports. Any horizontal or vertical misalignment of the joint elements should be recorded and checked at future inspections. Where drainage troughs are provided, check for a build-up of debris that prevents proper drainage and causes spill over onto the superstructure and substructure components, or impedes joint movement.

Sealed armored joints such as strip seals or compression seals should be checked for the presence of defects such as tears, separations, sagging, protrusions or embedment of foreign material. Ultraviolet degradation of the seal material is evidenced by hardening and brittleness of the surface and by the appearance of pattern cracking. The underside of all sealed deck joints should be checked for evidence of active joint leakage shown by water staining of the underlying structural elements. Areas of water staining should be clearly marked on drawings or in the field notes, so that future inspections can more accurately assess the extent of active leakage.

Reinforced elastomeric joints are composed of various proprietary combinations of steel supports and sealant material. Inspect for missing anchor bolt covers, separation of joint elements and audible or visual evidence of loose joint panels under traffic. Loose joint panels should be repaired immediately because the bolt failure is progressive and may result in one of the joint panels breaking loose under traffic.

Modular joints are composed of single or multiple support systems working together to accommodate large bridge movements. Inspect for surface damage to seals and separation beams. Examine underside for evidence of leakage and also for unusual noise which may indicate fractured welds or bolts.

3.8.4.6 Railings, Sidewalks and Curbs

3.8.4.6.1 Railings

Bridge railing and parapets, if present, should be evaluated as to condition and as to adequacy of geometry and structural capacity. The inspector should be familiar with the railing requirements of the Bridge Owner. On through-truss bridges, the structural elements, especially fracture critical members such as eyebars, hangers, etc., should be separated from traffic by an adequate vehicular railing system to prevent vehicle impact from causing major structural damage penetrant seeps from the crack where it is trapped and stains the developer. For this reason bright-colored (often red) penetrants are used. The red penetrant stains on the white chalky developer indicate the presence of a crack or other defect when visually inspected by the examiner. Modifications of the system include penetrants of different viscosity to detect different size cracks, wet rather than dry developers, and penetrants that fluoresce under ultraviolet light. These penetrants, used in conjunction with ultraviolet light examination, make smaller defects visible.

The principal advantages of the method are the ease with which the tests are conducted, the minimal skills required, and the low cost. Tests are not time consuming and may be made frequently during other operations (for example, to determine if a defect being removed by grinding is completely eliminated). It must be considered the most portable of all methods.

The principal disadvantage is that only surface defects can be detected. This places a limitation on the usefulness of the method for the defect depth determination and "code" approval of most structures. However, from the practical shop viewpoint, many defects that occur during construction (for example, weld cracks) are detectable if dye penetrant is used at intermediate stages in the construction. Thus, defects that are later buried can be detected and repaired before they are hidden from view. Use of dye penetrant during fabrication may prevent later rejection when ultrasonic or X-ray examination is used. The more sophisticated dye penetrant methods, using ultraviolet light, are rarely used in field applications.

4.2.2.5 Ultrasonic Examination

Ultrasonic testing relies on the wave properties of sound in materials to detect internal flaws. High-frequency sound waves in the form of mechanical vibrations are applied to the part to be tested and the waves, passing through the material, strike either a defect or, eventually, an external surface. The sound vibrations are then reflected and the nature of the return signal indicates the location and type of reflecting surface. Normal instrumentation includes a sound wave generator and pick-up device (usually combined in one unit) and a display screen on which the initial and reflected pulse is displayed. Display instrumentation permits an estimation of the position (in depth) of the defect, the nature of the defect and, by moving the detection portion of the unit (called the search unit) along the part to be examined, the size of the defect. The test sensitivity is influenced by a great number of testing variables, such as sound frequency, design of the search unit, instrumentation, electronic processing of the return signal, and the skill of the operator. Normal results of the examination are a form prepared by the operator based on his observations of the display screen.

The major advantages of this system of nondestructive examination are its portability, sensitivity and ability to detect the location of cracks or defects in depth. On the other hand, the major fault of the system is that, until very recent times, no permanent record of the defect was produced. It is now possible to make photographic records of the display, and equipment is now available to permit the storage of field data in a format suitable for subsequent computer processing and reporting. Another characteristic of the system often cited as a difficulty is the sensitivity of the method. It is possible to see too much; i.e., grain size in metals and minor defects not observable by other methods. The system cannot detect surface defects very well. The dependency of the method on operator skill must also be considered an unfavorable factor.

More research has been undertaken to modify this method and make it more widely applicable than most of the others, so advances in technology are more likely in this field.

4.2.3 Timber Field Tests

Typical field test procedures for detecting defects and deterioration in timber bridge components are described below.

A summary of the capabilities of each of the test methods for detecting defects and deterioration in timber components is given in Table 4.2.3. This table should be used as a guide in selecting an appropriate field test procedure for timber components.

4.2.3.1 Penetration Methods

Any probe, such as a knife, ice pick, nail, or brace and bit, can be used to test for internal decay or vermin infestation. The ease with which a member can be penetrated is then a measure of its soundness. Only a qualitative assessment is obtained because the pressure on the instrument is neither controlled nor measured. Although the procedure is rather crude, it is rapid and an overall assessment of the condition

Table 4.2.3 Capability of Investigative Techniques for Detecting Defects in Timber Structures in Field Use

Method Based on	Capability of Defect Detection[a]				
	Surface Decay and Rot	Internal Decay and Voids	Weathering	Chemical Attack	Abrasion and Wear
Penetration	G	G	F	F	N
Electrical	F	F	N	N	N
Ultrasonics	N	G	G	N	N

[a]G = Good; F = Fair; P = Poor; N = Not suitable.

of a structure can be obtained quickly. The use of a probe is much more satisfactory than attempting to identify a hollow member by sounding because the load on the member affects the response and may lead to erroneous conclusions.

An increment borer, which consists of a sharpened hollow tube, usually about 1/4-in. (6-mm) internal diameter, can also be used to penetrate the wood. The borer is superior to a nail or ice pick because it gives a more accurate record of the depth of decay or infestation. It also allows samples to be removed from the interior of the member for detailed examination or testing for such items as moisture content and preservative penetration, or to be cultured for positive evidence of decay fungi.

4.2.3.2 Electrical Methods

The main application of electrical methods is to measure the moisture content of timber. There are several electrical techniques available for measuring moisture content.

Resistance meters are based on a direct current measurement of electrical resistance between point or blade electrodes pushed into the timber. The resistance is related to the moisture content, which is displayed on a calibrated scale. The results are affected by the species of timber and correction factors must be applied. Resistance moisture meters are light, compact, and inexpensive but the major disadvantage is that they measure the moisture content of the surface layers unless special deep probes are used. Readings over 30 percent moisture content are not reliable and contamination by some chemicals, such as salt, affects the readings.

Capacitance meters are based on an alternating current measure of the dielectric constant of wood, which is proportional to its moisture content. The results are a function of the relative density of the wood and correction factors must be applied. The meters measure primarily surface moisture content, and, on lumber thicker than 2 in. (50 mm), do not respond to internal moisture adequately. Capacitance meters have a wider range (0 to at least 35 percent moisture content) than resistance meters and are less affected by the presence of chemicals.

Radio frequency power-loss meters operate in the frequency range 0 to 25 MHz and are based on an alternating current measurement of the impedance (combined effect of resistance) and capacitance of timber. They need to be calibrated for wood species and density. The meters use plate-type electrodes and the field penetrates about 3/4 in. (20 mm) but the surface layers have the predominant effect. The cost of the meters is similar to that of capacity-type meters, being higher than that of simple resistance types.

Electrical resistance measurements are also the basis of an instrument designed to detect internal rot. The device consists of a resistance probe, which is inserted to various depths in a hole 3/32 in. (2.4 mm) in diameter. A marked change in electrical resistance is an indication of decay. Although the device effectively detects rot, it is susceptible to false indications of decay in apparently sound wood.

4.2.3.3 Ultrasonic Techniques

The same ultrasonic pulse-velocity equipment and techniques described in Article 4.2.1.3 for application to concrete members can also be used for the in-situ testing of timber structures, both above and below the water surface.

Pulse-velocity measurements relate to the elastic properties of the wood and are therefore sensitive to the direction of the grain. However, pulse-velocity measurements have been found to follow similar trends to strength changes caused by fluctuations in density and local defects. Consequently, the strength and stiffness properties of the timber can be assessed. The ultrasonic method can also be used to identify internal decay and hollow areas, as well as internal knots, checks, and shakes. Because a discontinuity, such as a crack or a hollow area caused by decay, reflects part of the sound wave and changes the velocity of the transmitted wave, the technique is most sensitive to detecting defects that are oriented perpendicularly to the pulse. For this reason, the direct transmission mode with transducers on opposite faces of the member is generally the most useful configuration. However, in some situations, it may be necessary to investigate other relative positions of the transducers in order to produce a maximum response. To simplify interpretation of the results it is common practice to compare the pulse velocity from a suspected area of deterioration with that from an area known to be sound (measured using the same transducer configuration), thereby eliminating the need to measure the density of the timber. In all cases, a good contact between the transducer and the surface of the timber is essential. A light grease or glycerol are suitable for the coupling medium. Bentonite paste has also been found satisfactory.

4.3 MATERIAL SAMPLING

Tests which require the removal of material from the structure should be used only when a particular piece of information is desired, and only when the results can provide something useful in the overall evaluation of the bridge.

A few common material sampling standards are shown in Table 4.3-1. Samples should be removed from those areas of a bridge subjected to low stress levels as determined by the Engineer. An adequate number of samples should be obtained to provide results representative of the entire structure being evaluated. Normally, a minimum of three samples would be required.

The removal of material from a structure will leave a hole or void in one or more members. Repairs can be readily made to concrete, masonry and timber members. Repairs to steel members may be much more complex, particularly if welding is used, and should be carried out by experienced personnel. Care should be taken to minimize any residual stress resulting from the repair.

Table 4.3-1 Standard ASTM and AASHTO Methods for Material Sampling

Designation	Title
C 42/T 24	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
T 260	Sampling and Testing for Total Chloride Ion in Concrete Raw Materials
C 823	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions
A 610	Sampling Ferroalloys for Size (Before or After Shipment)
A 673	Sampling Procedures for Impact Testing of Structural Steel (Charpy Test)
A 370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products

TABLE 4.4-1 Standard ASTM and AASHTO Test Methods for Concrete for Use in the Laboratory

Designation[a]	Title
C 39/T 22	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 85/T 178	Test Method for Cement Content of Hardened Portland Cement Concrete
C 174/T 148	Method of Measuring Length of Drilled Concrete Cores
C 457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C 469	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C 496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 617/T 231	Method of Capping Cylindrical Concrete Specimens
C 642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
C 666/T 161	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 856	Recommended Practice for Petrographic Examination of Hardened Concrete
T 259	Method of Test for Resistance of Concrete to Chloride Ion Penetration[b]
T 260	Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials
T 277	Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete

[a]ASTM test methods are designated C. AASHTO test methods are designated T.
[b]Corrosion threshold is about 1.3 to 2.0 pounds of chloride per cubic yard.

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6. LOAD RATING

6.1 GENERAL

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load rating requires engineering judgment in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. Bridge load rating calculations are based on information in the bridge file including the results of a recent inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection.

Bridge Owners should implement standardized procedures for determining the load rating of bridges based on this Manual.

This Manual provides a choice of load rating methods. Load ratings at Operating and Inventory levels using the allowable stress method can be calculated and may be especially useful for comparison with past practices. Similarly, load ratings at Operating and Inventory levels based on the load factor method can also be calculated. Each of these rating methods is presented below.

In addition, some Bridge Owners may elect to determine the bridge rating by the load and resistance factor rating method (LRFR). This method is described in the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges*.

6.1.1 Assumptions

The safe load capacity of a bridge is based on existing structural conditions. To maintain this capacity, it is assumed that the bridges are subject to competent inspections as often as the existing conditions of the structures require, and that sound judgment will be exercised in determining an appropriate safety margin.

6.1.2 Substructure Consideration

Careful attention should be given to all elements of the substructure for evidence of instability which affects the load-carrying capacity of a bridge. Evaluation of the conditions of a bridge's substructure will in many cases be a matter of good engineering judgment.

The adequacy of the substructure should be based on information from as-built plans, construction plans, design calculations, inspection results and other appropriate data. When such information is available, the substructure elements, including piers and abutments, should be checked to ensure that they have at least the capacity of the lowest rated superstructure member. If such information is not available, the substructure should be assumed to be adequate if it is judged by the engineer to be stable after examining the alignment, condition and performance of the substructure elements over time.

6.1.3 Safety Criteria

In general, the safety factors to be used should be taken from this Manual. However, there are some cases where judgment must be exercised in making an evaluation of a structure and the safety factor may be adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. This determination most commonly applies to timber which may be of substandard grade or where the material is weathered or otherwise deteriorated. In determining the safety factor for a bridge, consideration should be given to the types of vehicles using the bridge routinely. Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public.

All data used in the determination of the safety factor should be fully documented.

6.1.4 Application of Standard Design Specifications

For all matters not covered by this Manual, the current applicable AASHTO *Standard Specifications for Highway Bridges* (AASHTO Design Specifications) should be used as a guide. However, there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented. Diagnostic

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)} \qquad (6\text{-}1a)$$

where:

- RF = the rating factor for the live-load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure (see equation 6-1b)
- C = the capacity of the member (see Article 6.6)
- D = the dead load effect on the member (see Article 6.7.1). For composite members, the dead load effect on the noncomposite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used
- L = the live load effect on the member (see Article 6.7.2)
- I = the impact factor to be used with the live load effect (see Article 6.7.4)
- A_1 = factor for dead loads (see Articles 6.5.2 and 6.5.3)
- A_2 = factor for live load (see Articles 6.5.2 and 6.5.3)

In the equation above "load effect" is the effect of the applied loads on the member. Typical "load effects" used by engineers are axial force, vertical shear force, bending moment, axial stress, shear stress and bending stresses. Once the "load effect" to be evaluated is selected by the engineer, the "capacity" of a member to resist such a load effect may be determined (see Article 6.6).

The Rating Factor (RF) may be used to determine the rating of the bridge member in tons as follows:

$$RT = (RF)W \qquad (6\text{-}1b)$$

where:

- RT = bridge member rating in tons
- W = weight (tons) of nominal truck used in determining the live load effect (L)

The rating of a bridge is controlled by the member with the lowest rating in tons.

6.5.2 Allowable Stress

For the allowable stress method, $A_1 = 1.0$ and $A_2 = 1.0$ in the general rating equation.

The capacity (C) depends on the rating level desired, with the higher value for "C" used for the Operating level. The determination of the nominal capacity of a member is discussed in Article 6.6.2.

6.5.3 Load Factor

For the load factor method, $A_1 = 1.3$ and A_2 varies depending on the rating level desired. For Inventory level, $A_2 = 2.17$ and for Operating level, $A_2 = 1.3$.

The nominal capacity (C) is the same regardless of the rating level desired (see Article 6.6.3).

6.5.4 Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach, and for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined as discussed in Section 3. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

Size, number, and relative location of bolts and rivets through tension members should be determined and recorded so that the net area of the section can be calculated. Also, in addition to the physical condition, threaded members such as truss rods at turn-buckles should be checked to see if the rod has been upset so that the net area will be properly calculated. This information will normally be taken from plans when they are available, but should be determined in the field otherwise. Any misalignment, bends, or kinks in compression members should be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, examine the connections of compression members carefully to see if they are

6.6.2.7 Timber

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the engineer making the field investigation.

(1) Inventory Stress
The Inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Design Specifications.

Allowable Inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Design Specifications.

(2) Operating Stress
The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Design Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable Operating stress in pounds per square inch of cross-sectional area of simple solid columns should be determined by the following formulae but the allowable Operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Design Specifications.

$$\frac{P}{A} = \frac{4.8E}{(l/r)^2} \tag{6-6}$$

in which

P = total load in pounds
A = cross-sectional area in square inches
E = modulus of elasticity
l = unsupported overall length, in inches, between points of lateral support of simple columns
r = least radius of gyration of the section in inches

For columns of square or rectangular cross section, this formula becomes:

$$\frac{P}{A} = \frac{0.40E}{(l/d)^2} \tag{6-7}$$

in which d = dimension in inches of the narrowest face.

The above formula applies to long columns with (l/d) over 11, but not greater than 50.

For short columns, (l/d) not over 11, use the allowable design unit stress in compression parallel to grain times 1.33 for the grade of timber used.

6.6.3 Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Design Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Design Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

6.6.3.1 Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "Date Built" column of the tables set forth in Article 6.6.2.1.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Design Specifications. The capacity (C) for typical steel bridge members is summarized in Appendix C. For beams, the overload limitations of Article 10.57 should also be considered.

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Design Specifications.

The Operating rating for friction joint fasteners (A 325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

2002 Interim Revisions
to the
STANDARD SPECIFICATIONS
FOR
HIGHWAY BRIDGES

SIXTEENTH EDITION

1996

Developed by the
AASHTO Subcommittee on Bridges and Structures

Published by the
American Association of State Highway
and Transportation Officials

From: 2002
AASHTO STD. SPECS.

Section 3
LOADS

Part A
TYPES OF LOADS

3.1 NOTATIONS

A = maximum expected acceleration of bedrock at the site
a = length of short span of slab (Article 3.24.6)
B = buoyancy (Article 3.22)
b = width of pier or diameter of pile (Article 3.18.2.2.4)
b = length of long span of slab (Article 3.24.6)
C = combined response coefficient
C = stiffness parameter = K(W/L) (Article 3.23.4.3)
C = centrifugal force in percent of live load (Article 3.10.1)
CF = centrifugal force (Article 3.22)
C_n = coefficient for nose inclination (Article 3.18.2.2.1)
C_M = steel bending stress coefficient (Article 3.25.1.5)
C_R = steel shear stress coefficient (Article 3.25.1.5)
D = parameter used in determination of load fraction of wheel load (Article 3.23.4.3)
D = degree of curve (Article 3.10.1)
D = dead load (Article 3.22)
D.F. = fraction of wheel load applied to beam (Article 3.28.1)
DL = contributing dead load
E = width of slab over which a wheel load is distributed (Article 3.24.3)
E = earth pressure (Article 3.22)
EQ = equivalent static horizontal force applied at the center of gravity of the structure
E_c = modulus of elasticity of concrete (Article 3.26.3)
E_s = modulus of elasticity of steel (Article 3.26.3)
E_w = modulus of elasticity of wood (Article 3.26.3)
F = horizontal ice force on pier (Article 3.18.2.2.1)
F = framing factor (Article 3.21.1.1)
F_b = allowable bending stress (Article 3.25.1.3)
F_v = allowable shear stress (Article 3.25.1.3)
g = 32.2 ft./sec.^{2}
I = impact fraction (Article 3.8.2)
I = gross flexural moment of inertia of the precast member (Article 3.23.4.3)
ICE = ice pressure (Article 3.22)
J = gross Saint-Venant torsional constant of the precast member (Article 3.23.4.3)
K = stream flow force constant (Article 3.18.1)
K = stiffness constant (Article 3.23.4)
K = wheel load distribution constant for timber flooring (Article 3.25.1.3)
k = live load distribution constant for spread box girders (Article 3.28.1)
L = loaded length of span (Article 3.8.2)
L = loaded length of sidewalk (Article 3.14.1.1)

L = live load (Article 3.22)
L = span length (Article 3.23.4)
LF = longitudinal force from live load (Article 3.22)
M_D = moment capacity of dowel (Article 3.25.1.4)
$\underline{M_x}$ = primary bending moment (Article 3.25.1.3)
$\underline{M_y}$ = total transferred secondary moment (Article 3.25.1.4)
N_B = number of beams (Article 3.28.1)
N_L = number of traffic lanes (Article 3.23.4)
n = number of dowels (Article 3.25.1.4)
P = live load on sidewalk (Article 3.14.1.1)
P = stream flow pressure (Article 3.18.1)
P = total uniform force required to cause unit horizontal deflection of whole structure
P = load on one rear wheel of truck (Article 3.24.3)
P = wheel load (Article 3.24.5)
P = design wheel load (Article 3.25.1.3)
P_{15} = 12,000 pounds (Article 3.24.3)
P_{20} = 16,000 pounds (Article 3.24.3)
p = effective ice strength (Article 3.18.2.2.1)
p = proportion of load carried by short span (Article 3.24.6.1)
R = radius of curve (Article 3.10.1)
R = normalized rock response
R = rib shortening (Article 3.22)
R_D = shear capacity of dowel (Article 3.25.1.4)
$\underline{R_x}$ = primary shear (Article 3.25.1.3)
$\underline{R_y}$ = total secondary shear transferred (Article 3.25.1.4)
S = design speed (Article 3.10.1)
S = soil amplification spectral ratio
S = shrinkage (Article 3.22)
S = average stringer spacing (Article 3.23.2.3.1)
S = spacing of beams (Article 3.23.3)
S = width of precast member (Article 3.23.4.3)
S = effective span length (Article 3.24.1)
S = span length (Article 3.24.8.2)
S = beam spacing (Article 3.28.1)
s = effective deck span (Article 3.25.1.3)
SF = stream flow (Article 3.22)
T = period of vibration
T = temperature (Article 3.22)
t = thickness of ice (Article 3.18.2.2.4)
t = deck thickness (Article 3.25.1.3)
V = variable spacing of truck axles (Figure 3.7.7A)
V = velocity of water (Article 3.18.1)
W = combined weight on the first two axles of a standard HS Truck (Figure 3.7.7A)
W = width of sidewalk (Article 3.14.1.1)
W = wind load on structure (Article 3.22)
W = total dead weight of the structure
W_e = width of exterior girder (Article 3.23.2.3.2)
W = overall width of bridge (Article 3.23.4.3)
W = roadway width between curbs (Article 3.28.1)
WL = wind load on live load (Article 3.22)
w = width of pier or diameter of circular-shaft pier at the level of ice action (Article 3.18.2.2.1)
X = distance from load to point of support (Article 3.24.5.1)
x = subscript denoting direction perpendicular to longitudinal stringers (Article 3.25.1.3)

3.3.5 Where the abrasion of concrete is not expected, the traffic may bear directly on the concrete slab. If considered desirable, ¼ inch or more may be added to the slab for a wearing surface.

3.3.6 The following weights are to be used in computing the dead load:

	#/cu.ft.
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (treated or untreated)	50
Concrete, plain or reinforced	150
Compacted sand, earth, gravel, or ballast	120
Loose sand, earth, and gravel	100
Macadam or gravel, rolled	140
Cinder filling	60
Pavement, other than wood block	150
Railway rails, guardrails, and fastenings (per linear foot of track)	200
Stone masonry	170
Asphalt plank, 1 in. thick	9 lb. sq. ft.

3.4 LIVE LOAD

The live load shall consist of the weight of the applied moving load of vehicles, cars, and pedestrians.

3.5 OVERLOAD PROVISIONS

3.5.1 For all loadings less than H 20, provision shall be made for an infrequent heavy load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures.

3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination Group IB in Article 3.22.

3.6 TRAFFIC LANES

3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet.

3.6.2 These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width measured between curbs.

3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions within their individual traffic lanes, so as to produce the maximum stress in the member under consideration.

3.7 HIGHWAY LOADS

3.7.1 Standard Truck and Lane Loads*

3.7.1.1 The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loading are provided, the H loadings and the HS loadings—the HS loadings being heavier than the corresponding H loadings.

3.7.1.2 Each lane load shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans—see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane.

3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses.

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*Note: The system of lane loads defined here (and illustrated in Figure 3.7.6.B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.

Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings.

In 1944, the HS series of trucks was developed. These approximate the effect of the corresponding 1935 truck preceded and followed by a train of trucks weighing three-fourths as much as the basic truck.

3.7.2 Classes of Loading

There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75 percent of loading H 20. Loading HS 15 is 75 percent of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads.

3.7.3 Designation of Loadings

The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 Edition in the following manner:

H 15 Loading, 1944 Edition shall be designated	H 15-44
H 20 Loading, 1944 Edition shall be designated	H 20-44
H 15-S 12 Loading, 1944 Edition shall be designated	HS 15-44
H 20-S 16 Loading, 1944 Edition shall be designated	HS 20-44

The affix shall remain unchanged until such time as the loading specification is revised. The same policy for identification shall be applied, for future reference, to loadings previously adopted by the American Association of State Highway and Transportation Officials.

3.7.4 Minimum Loading

Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy truck traffic, shall be designed for HS20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress.

3.7.5 H Loading

The H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck.

3.7.6 HS Loading

The HS loadings consist of a tractor truck with semitrailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.6B. The HS loadings are designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments.

3.8 IMPACT

3.8.1 Application

Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground.

3.8.1.1 Group A—Impact shall be included.

(1) Superstructure, including legs of rigid frames.
(2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line.
(3) The portions above the ground line of concrete or steel piles that support the superstructure.

3.8.1.2 Group B—Impact shall not be included.

(1) Abutments, retaining walls, piles except as specified in 3.8.1.1 (3).
(2) Foundation pressures and footings.
(3) Timber structures.
(4) Sidewalk loads.
(5) Culverts and structures having 3 feet or more cover.

3.8.2. Impact Formula

3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula:

$$I = \frac{50}{L+125} \qquad (3\text{-}1)$$

in which,

I = impact fraction (maximum 30 percent);

3.10 CENTRIFUGAL FORCES

3.10.1 Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes:

$$C = 0.00117S^2D = \frac{6.68S^2}{R} \qquad (3\text{-}2)$$

where,

C = the centrifugal force in percent of the live load, without impact;
S = the design speed in miles per hour;
D = the degree of curve;
R = the radius of the curve in feet.

3.10.2 The effects of superelevation shall be taken into account.

3.10.3 The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 3.7 with one standard truck on each design traffic lane placed in position for maximum loading.

3.10.4 Lane loads shall not be used in the computation of centrifugal forces.

3.10.5 When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load.

3.11 APPLICATION OF LIVE LOAD

3.11.1 Traffic Lane Units

In computing stresses, each 10-foot lane load or single standard truck shall be considered as a unit, and fractions of load lane widths or trucks shall not be used.

3.11.2 Number and Position of Traffic Lane Units

The number and position of the lane load or truck loads shall be as specified in Article 3.7 and, whether lane or truck loads, shall be such as to produce maximum stress, subject to the reduction specified in Article 3.12.

3.11.3 Lane Loads on Continuous Spans

For the determination of maximum negative moment in the design of continuous spans, the lane load shown in Figure 3.7.6B shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment.

3.11.4 Loading for Maximum Stress

3.11.4.1 On both simple and continuous spans, the type of loading, whether lane load or truck load, to be used shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which types of loading controls for simple spans.

3.11.4.2 For continuous spans, the lane loading shall be continuous or discontinuous; only one standard H or HS truck per lane shall be considered on the structure.

3.12 REDUCTION IN LOAD INTENSITY

3.12.1 Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, the following percentages of the live loads may be used in view of the improbability of coincident maximum loading:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.12.2 The reduction in load intensity specified in Article 3.12.1 shall not be applicable when distribution factors from Table 3.23.1 are used to determine moments in longitudinal beams.

3.12.3 The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam.

TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
Timber:[a]		
Plank[b]	S/4.0	S/3.75
Nail laminated[c] 4" thick or multiple layer[d] floors over 5" thick	S/4.5	S/4.0
Nail laminated[c] 6" or more thick	S/5.0 If S exceeds 5′ use footnote f.	S/4.25 If S exceeds 6.5′ use footnote f.
Glued laminated[e] Panels on glued laminated stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 7.5′ use footnote f.
On steel stringers		
4" thick	S/4.5	S/4.0
6" or more thick	S/5.25 If S exceeds 5.5′ use footnote f.	S/4.5 If S exceeds 7′ use footnote f.
Concrete:		
On steel I-Beam stringers[g] and prestressed concrete girders	S/7.0 If S exceeds 10′ use footnote f.	S/5.5 If S exceeds 14′ use footnote f.
On concrete T-Beams	S/6.5 If S exceeds 6′ use footnote f.	S/6.0 If S exceeds 10′ use footnote f.
On timber stringers	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 10′ use footnote f.
Concrete box girders[h]	S/8.0 If S exceeds 12′ use footnote f.	S/7.0 If S exceeds 16′ use footnote f.
On steel box girders	See Article 10.39.2.	
On prestressed concrete spread box Beams	See Article 3.28.	
Steel grid:		
(Less than 4" thick)	S/4.5	S/4.0
(4" or more)	S/6.0 If S exceeds 6′ use footnote f.	S/5.0 If S exceeds 10.5′ use footnote f.
Steel bridge Corrugated plank[i] (2" min. depth)	S/5.5	S/4.5

S = average stringer spacing in feet.

[a]Timber dimensions shown are for nominal thickness.

[b]Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11—Division II).

[c]Nail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed to the preceding piece (see Article 16.3.12—Division II).

[d]Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other (see Article 16.3.11—Division II).

[e]Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13—Division II).

[f]In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.

[g]"Design of I-Beam Bridges" by N. M. Newmark—Proceedings, ASCE, March 1948.

[h]The sidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

[i]Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:

Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland.

These distribution factors were developed based on studies using 6" × 2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" × 2" corrugated plank used in the studies.

3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of construction are different from those anticipated by the specifications.

3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H 20, Group IA loading combination shall be used (see Article 3.5).

Part C
DISTRIBUTION OF LOADS

3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS*

3.23.1 Position of Loads for Shear

3.23.1.1 In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the transverse floor beam or the end of the longitudinal beam or stringer at which the stress is being determined.

3.23.1.2 Lateral distribution of the wheel loads at ends of the beams or stringers shall be that produced by

*Provisions in this Article shall not apply to orthotropic deck bridges.

assuming the flooring to act as a simple span between stringers or beams. For wheels or axles in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the calculations of horizontal shear in rectangular timber beams shall be in accordance with Article 13.3.

3.23.2 Bending Moments in Stringers and Longitudinal Beams**

3.23.2.1 General

In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.

3.23.2.2 Interior Stringers and Beams

The live load bending moment for each interior stringer shall be determined by applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1.

3.23.2.3 Outside Roadway Stringers and Beams

3.23.2.3.1 Steel-Timber-Concrete T-Beams

3.23.2.3.1.1 The dead load supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surface, if placed after the slab has cured, may be distributed equally to all roadway stringers or beams.

3.23.2.3.1.2 The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

3.23.2.3.1.3 When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact and the structure is to be designed by the service load method, the allowable stress in the beam or stringer may be increased by 25 percent for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When the combination of sidewalk live load and traffic live load plus impact governs the design and the structure is to be designed by the load factor method, 1.25 may be used as the beta factor in place of 1.67.

**In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges.

3.23.2.3.1.4 In no case shall an exterior stringer have less carrying capacity than an interior stringer.

3.23.2.3.1.5 In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than:

$$\frac{S}{5.5}$$

where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and

$$\frac{S}{4.0 + 0.25S}$$

where, S is more than 6 feet and less than 14 feet. When S is 14 feet or more, use footnote f, Table 3.23.1.

3.23.2.3.2 Concrete Box Girders

3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1.

3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be $W_e/7$, where W_e is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.

3.23.2.3.3 Total Capacity of Stringers and Beams

The combined design load capacity of all the beams and stringers in a span shall not be less than required to support the total live and dead load in the span.

3.23.3 Bending Moments in Floor Beams (Transverse)

3.23.3.1 In calculating bending moments in floor beams, no transverse distribution of the wheel loads shall be assumed.

3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1.

where,

p = proportion of load carried by short span;
a = length of short span of slab;
b = length of long span of slab.

3.24.6.2 Where the length of the slab exceeds 1½ times its width, the entire load shall be carried by the transverse reinforcement.

3.24.6.3 The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. The moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced by 50 percent. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams.

3.24.7 Median Slabs

Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live, and impact stresses shall not be greater than 150 percent of the allowable stresses. Flush median slabs shall be designed without overstress.

3.24.8 Longitudinal Edge Beams

3.24.8.1 Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb.

3.24.8.2 The edge beam of a simple span shall be designed to resist a live load moment of 0.10 PS, where,

P = wheel load in pounds P_{15} or P_{20};
S = span length in feet.

3.24.8.3 For continuous spans, the moment may be reduced by 20 percent unless a greater reduction results from a more exact analysis.

3.24.9 Unsupported Transverse Edges

The design assumptions of this article do not provide for the effect of loads near unsupported edges. Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them.

3.24.10 Distribution Reinforcement

3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds 2 feet.

3.24.10.2 The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas:

For main reinforcement parallel to traffic,

$$\text{Percentage} = \frac{100}{\sqrt{S}} \text{ Maximum } 50\% \qquad (3\text{-}21)$$

For main reinforcement perpendicular to traffic,

$$\text{Percentage} = \frac{220}{\sqrt{S}} \text{ Maximum } 67\% \qquad (3\text{-}22)$$

where, S = the effective span length in feet.

3.24.10.3 For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50 percent of the specified amount shall be used in the outer quarters of the slab span.

3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING

For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows.

3.25.1 Transverse Flooring

3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30.

Normal to the direction of flooring span, the wheel load shall be distributed as follows:

Plank floor: the width of plank, but not less than 10 inches.

Non-interconnected* nail laminated panel floor: 15 inches, but not to exceed panel width.

Non-interconnected glued laminated panel floor: 15 inches plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated panel floor, with adequate shear transfer between panels**: 15 inches plus thickness of floor, but not to exceed panel width.

Interconnected* glued laminated panel floor, with adequate shear transfer between panels**, not less than 6 inches thick: 15 inches plus twice thickness of floor, but not to exceed panel width.

3.25.1.2 For transverse flooring the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness.

3.25.1.3 One design method for interconnected glued laminated panel floors is as follows: For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers and with panels interconnected using steel dowels, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear.† The maximum shear is for a wheel position assumed to be 15 inches or less from the center line of the support. The maximum moment is for a wheel position assumed to be centered between the supports.

$$M_x = P(.51 \log_{10} s - K) \qquad (3\text{-}23)$$

$$R_x = .034P \qquad (3\text{-}24)$$

Thus,

$$t = \sqrt{\frac{6M_x}{F_b}} \qquad (3\text{-}25)$$

or,

$$t = \frac{3R_x}{2F_v} \textit{ whichever is greater} \qquad (3\text{-}26)$$

where,

M_x = primary bending moment in inch-pounds per inch;
R_x = primary shear in pounds per inch;
x = denotes direction perpendicular to longitudinal stringers;
P = design wheel load in pounds;
s = effective deck span in inches;
t = deck thickness, in inches, based on moment or shear, whichever controls;
K = design constant depending on design load as follows:

H 15	K = 0.47
H 20	K = 0.51

F_b = allowable bending stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B);
F_v = allowable shear stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2A and B).

3.25.1.4 The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels shall be based on the following equation:

$$n = \frac{1{,}000}{\sigma_{PL}} \times \left[\frac{\overline{R_y}}{R_D} + \frac{\overline{M_y}}{M_D}\right] \qquad (3\text{-}27)$$

where,

n = number of steel dowels required for the given spans;
σ_{PL} = proportional limit stress perpendicular to grain (for Douglas Fir or Southern pine, use 1,000 psi);
$\overline{R_y}$ = total secondary shear transferred, in pounds, determined by the relationship:

$$\overline{R_y} = 6Ps/1{,}000 \text{ for } s \leq 50 \text{ inches} \qquad (3\text{-}28)$$

or,

$$\overline{R_y} = \frac{P}{2s}(s-20) \text{ for } s > 50 \text{ inches} \qquad (3\text{-}29)$$

$\overline{M_y}$ = total secondary moment transferred, in inch-pound, determined by the relationship,

$$\overline{M_y} = \frac{Ps}{1{,}600}(s-10) \text{ for } s \leq 50 \text{ inches} \qquad (3\text{-}30)$$

$$\overline{M_y} = \frac{Ps}{20}\frac{(s-30)}{(s-10)} \text{ for } s > 50 \text{ inches} \qquad (3\text{-}31)$$

R_D and M_D = shear and moment capacities, respectively, as given in the following table:

*The terms interconnected and non-interconnected refer to the joints between the individual nail laminated or glued laminated panels.

**This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or other suitable means.

†The equations are developed for deck panel spans equal to or greater than the width of the tire (as specified in Article 3.30), but not greater than 200 inches.

Diameter of Dowel	Shear Capacity R_D	Moment Capacity M_D	Steel Stress Coefficients C_R	C_M	Total Dowel Length Required
in.	lb.	in.-lb.	$1/in.^2$	$1/in.^3$	in.
0.5	600	850	36.9	81.5	8.50
.625	800	1,340	22.3	41.7	10.00
.75	1,020	1,960	14.8	24.1	11.50
.875	1,260	2,720	10.5	15.2	13.00
1.0	1,520	3,630	7.75	10.2	14.50
1.125	1,790	4,680	5.94	7.15	15.50
1.25	2,100	5,950	4.69	5.22	17.00
1.375	2,420	7,360	3.78	3.92	18.00
1.5	2,770	8,990	3.11	3.02	19.50

3.25.1.5 In addition, the dowels shall be checked to ensure that the allowable stress of the steel is not exceeded using the following equation:

$$\sigma = \frac{1}{n}(C_R \overline{R_y} + C_M \overline{M_y}) \qquad (3\text{-}32)$$

where,

σ = minimum yield point of steel pins in pounds per square inch (see Table 10.32.1A);

$n, \overline{R_y}, \overline{M_y}$ = as previously defined;

C_R, C_M = steel stress coefficients as given in preceding table.

3.25.2 Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be distributed over 10 inches.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: 20 inches;

Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated floor, with adequate shear transfer between panels*, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

*This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or spreader beams located at intervals along the panels or other suitable means.

3.25.3 Longitudinal Glued Laminated Timber Decks

3.25.3.1 Bending Moment

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

$$\text{Load Fraction} = \frac{W_p}{3.75 + \frac{L}{28}} \text{ or } \frac{W_p}{5.00}, \text{ whichever is}$$

greater.

ONE TRAFFIC LANE

$$\text{Load Fraction} = \frac{W_p}{4.25 + \frac{L}{28}} \text{ or } \frac{W_p}{5.50}, \text{ whichever is}$$

greater.

where, W_p = Width of Panel; in feet ($3.5 \le W_p \le 4.5$)

L = Length of span for simple span bridges and the length of the shortest span for continuous bridges in feet.

3.25.3.2 Shear (Glue Laminated)

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.2 Shear

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 Deflections

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 Stiffener Arrangement

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at mid-span with additional stiffeners placed at intervals not to exceed 10 feet. The stiffness factor EI of the stiffener shall not be less than 80,000 kip-in^2.

3.25.4 Continuous Flooring

If the flooring is continuous over more than two spans, the maximum bending moment shall be assumed as being 80% of that obtained for a simple span.

3.26 DISTRIBUTION OF WHEEL LOADS AND DESIGN OF COMPOSITE WOOD-CONCRETE MEMBERS

3.26.1 Distribution of Concentrated Loads for Bending Moment and Shear

3.26.1.1 For freely supported or continuous slab spans of composite wood-concrete construction, as described in Article 16.3.14, Division II, the wheel loads shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear.

3.26.1.2 For composite T-beams of wood and concrete, as described in Article 20.19.2, Division II, the effective flange width shall not exceed that given in Article 10.38.3. Shear connectors shall be capable of resisting both vertical and horizontal movement.

3.26.2 Distribution of Bending Moments in Continuous Spans

3.26.2.1 Both positive and negative moments shall be distributed in accordance with the following table:

Maximum Bending Moments—Percent of Simple Span Moment

	Maximum Uniform Dead Load Moments				Maximum Live Load Moments			
	Wood Subdeck		Composite Slab		Concentrated Load		Uniform Load	
Span	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Interior	50	50	55	45	75	25	75	55
End	70	60	70	60	85	30	85	65
2-Span[a]	65	70	60	75	85	30	80	75

[a]Continuous beam of 2 equal spans.

3.26.2.2 Impact should be considered in computing stresses for concrete and steel, but neglected for wood.

3.26.3 Design

The analysis and design of composite wood-concrete members shall be based on assumptions that account for the different mechanical properties of the components. A suitable procedure may be based on the elastic properties of the materials as follows:

$\frac{E_c}{E_w}$ = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section

$\frac{E_c}{E_w}$ = 2 for slab in which the net concrete thickness is at least half the overall depth of the composite section

$\frac{E_s}{E_w}$ = 18.75 (for Douglas fir and Southern pine)

in which,

E_c = modulus of elasticity of concrete;
E_w = modulus of elasticity of wood;
E_s = modulus of elasticity of steel.

3.27 DISTRIBUTION OF WHEEL LOADS ON STEEL GRID FLOORS*

3.27.1 General

3.27.1.1 The grid floor shall be designed as continuous, but simple span moments may be used and reduced as provided in Article 3.24.

3.27.1.2 The following rules for distribution of loads assume that the grid floor is composed of main elements that span between girders, stringers, or cross beams, and secondary elements that are capable of transferring load between the main elements.

3.27.1.3 Reinforcement for secondary elements shall consist of bars or shapes welded to the main steel.

3.27.2 Floors Filled with Concrete

3.27.2.1 The distribution and bending moment shall be as specified for concrete slabs, Article 3.24. The following items specified in that article shall also apply to concrete filled steel grid floors:

Longitudinal edge beams
Unsupported transverse edges
Span lengths

3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 8.15.2, 8.16.1, and 10.32.

3.27.3 Open Floors

3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1¼ inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H 20, 15 inches for H 15).

3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32.

*Provisions in this article shall not apply to orthotropic bridge superstructures.

3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly.

3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS**

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$\text{D.F.} = \frac{2N_L}{N_B} + k\frac{S}{L} \qquad (3\text{-}33)$$

where,

N_L = number of design traffic lanes (Article 3.6);
N_B = number of beams ($4 \leq N_B \leq 10$);
S = beam spacing in feet ($6.57 \leq S \leq 11.00$);
L = span length in feet;
k = $0.07\,W - N_L\,(0.10N_L - 0.26) - 0.20N_B - 0.12$; (3-34)
W = numeric value of the roadway width between curbs expressed in feet ($32 \leq W \leq 66$).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

3.29 MOMENTS, SHEARS, AND REACTIONS

Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress.

**The provisions of Article 3.12, Reduction in Load Intensity, were not applied in the development of the provisions presented in Articles 3.28.1 and 3.28.2.

3.30 TIRE CONTACT AREA

The tire contact area for the Alternate Military Loading or HS 20-44 shall be assumed as a rectangle with a length in the direction of traffic of 10 inches, and a width of tire of 20 inches. For other design vehicles, the tire contact should be determined by the engineer.

The following items specified in that article shall also apply to concrete filled steel grid floors:

Longitudinal edge beams
Unsupported transverse edges
Span lengths

3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forth in Articles 8.15.2, 8.16.1, and 10.32.

3.27.3 Open Floors

3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1 1/4 inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H20, 15 inches for H15).

3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32.

3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly.

3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS*

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k\frac{S}{L} \qquad (3\text{-}33)$$

where

N_L = number of design traffic lanes (Article 3.6);
N_B = number of beams ($4 \leq N_B \leq 10$);
S = beam spacing in feet ($6.57 \leq S \leq 11.00$);
L = span length in feet;
k = $0.07\,W - N_L\,(0.10N_L - 0.26) - 0.20N_B - 0.12$; (3-34)
W = numeric value of the roadway width between curbs expressed in feet ($32 \leq W \leq 66$).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than $2N_L/N_B$.

3.29 MOMENTS, SHEARS, AND REACTIONS

Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress.

3.30 TIRE CONTACT AREA

The tire contact area shall be assumed as a rectangle with an area in square inches of 0.01P, and a Length in Direction of Traffic/Width of Tire ratio of 1/2.5, in which P = wheel load in pounds.

FROM AASHTO 1986, USE FOR OTHER DESIGN VEHICLES.

*The provisions of Article 3.12, Reduction in Load Intensity, were not applied in the development of the provisions presented in 3.28.1 and 3.28.2.

From: 2002
AASHTO STD. SPECS

Section 13
WOOD STRUCTURES

13.1 GENERAL AND NOTATIONS

13.1.1 General

The following information on wood design is generally based on the National Design Specification for Wood Construction (NDS®), 1991 Edition. See the 1991 Edition of the NDS® for additional information.

13.1.2 Net Section

In determining the capacity of wood members, the net section of the member shall be used. Unless otherwise noted, the net section shall be determined by deducting from the gross section, the projected area of all material removed by boring, grooving, dapping, notching or other means.

13.1.3 Impact

In calculating live load stresses in wood, impact shall be neglected unless otherwise noted. See Article 3.8.1.

13.1.4 Notations

a = coefficient based on support conditions for tapered columns (Article 13.7.3.4.2)
b = width of bending member (Article 13.6.4.3)
c = coefficient based on sawn lumber, round timber piles, glued laminated timber or structural composite lumber (Article 13.7.3.3.5)
C_D = load duration factor (Article 13.5.5.2)
C_F = bending size factor for sawn lumber, structural composite lumber, and for glued laminated timber with loads applied parallel to the wide face of the laminations (Article 13.6.4.2)
C_F = compression size factor for sawn lumber (footnotes to Table 13.5.1A)
C_F = tension size factor for sawn lumber (footnotes to Table 13.5.1A) and structural composite lumber (footnotes to Tables 13.5.4A and 13.5.4B)
C_H = sheer stress factor (footnotes to Table 13.5.1A)
C_L = beam stability factor (Article 13.6.4.4)
C_M = wet service factor (Article 13.5.5.1)
C_P = column stability factor (Article 13.7.3.3)
C_V = volume factor for glued laminated timber with loads applied perpendicular to the wide face of the laminations (Article 13.6.4.3)
C_b = bearing area factor (Article 13.6.6.3)
C_f = form factor (Article 13.6.4.5)
C_{fu} = flat use factor for sawn lumber (footnotes to Table 13.5.1A)
C_r = repetitive member factor for sawn lumber (footnotes to Table 13.5.1A)
d = depth of member (Article 13.6.4.2.2)
d_{max} = maximum column face dimension (Article 13.7.3.4.2)
d_{min} = minimum column face dimension (Article 13.7.3.4.2)
d_{rep} = representative dimension for a tapered column face (Article 13.7.3.4.2)
E = tabulated modulus of elasticity (Article 13.6.3)
E′ = allowable modulus of elasticity (Article 13.6.3)
F_b = tabulated unit stress in bending (Article 13.6.4.1)
F_b' = allowable unit stress in bending (Article 13.6.4.1)
F_b^* = adjusted tabulated bending stress for beam stability (Article 13.6.4.4.5)
F_c = tabulated unit stress in compression parallel to grain (Article 13.7.3.2)
F_c' = allowable unit stress in compression parallel to grain (Article 13.7.3.2)
F_c^* = adjusted tabulated stress in compression parallel to grain for column stability (Article 13.7.3.3.5)
f_c = actual unit stress in compression parallel to grain (Article 13.7.3.1)
$F_{c\perp}$ = tabulated unit stress in compression perpendicular to grain (Article 13.6.6.2)
$F_{c\perp}'$ = allowable unit stress in compression perpendicular to grain (Article 13.6.6.2)
F_g = tabulated unit stress in bearing parallel to grain (Article 13.7.4.1)
F_g' = allowable unit stress in bearing parallel to grain (Article 13.7.4.1)

F_t = tabulated unit stress in tension parallel to grain (Article 13.8.1)
F_t' = allowable unit stress in tension parallel to grain (Article 13.8.1)
F_v = tabulated unit stress in shear parallel to grain (Article 13.6.5.3)
F_v' = allowable unit stress in shear parallel to grain (Article 13.6.5.3)
f_v = actual unit stress in shear parallel to grain (Article 13.6.5.2)
F_{θ}' = allowable unit stress for bearing on an inclined surface (Article 13.6.7)
K = column effective length factor (Article 13.7.3.3.3)
K_{bE} = material factor for beam stability (Article 13.6.4.4.5)
K_{cE} = material factor for column stability (Article 13.7.3.3.5)
L = length of bending member between points of zero moment (Article 13.6.4.3.1)
l = actual column length between points of lateral support (Article 13.7.3.3.3)
l_b = length of bearing (Article 13.6.6.3)
l_e = effective bending member length (Article 13.6.4.4.3)
l_e = effective column length (Article 13.7.3.3.3)
l_u = unsupported bending member length (Article 13.6.4.4.3)
m = parameter for the specific material determined in accordance with the requirements of ASTM D-5456 (Tables 13.5.4A and 13.5.4B)
R_B = bending member slenderness ratio (Article 13.6.4.4.4)
V = vertical shear (Article 13.6.5.2)
V_{LD} = maximum vertical shear at 3d or L/4 due to wheel loads distributed laterally as specified for moment (Article 13.6.5.2)
V_{LL} = distributed live load vertical shear (Article 13.6.5.2)
V_{LU} = maximum vertical shear at 3d or L/4 due to undistributed wheel loads (Article 13.6.5.2)
x = species variable for computing the volume factor (Article 13.6.4.3.1)
θ = angle between the direction of load and the direction of grain (Article 13.6.7)

13.2 MATERIALS

13.2.1 Sawn Lumber

13.2.1.1 General

Sawn lumber shall comply with the requirements of AASHTO M 168.

13.2.1.2 Dimensions

13.2.1.2.1 Structural calculations for sawn lumber shall be based on the net dimensions of the member for the anticipated use conditions. These net dimensions depend on the type of surfacing, whether dressed, rough-sawn or full-sawn.

13.2.1.2.2 For dressed lumber, the net dry dimensions given in Table 13.2.1A shall be used for design, regardless of the moisture content at the time of manufacture or in use.

13.2.1.2.3 Where the design is based on rough, full-sawn or special sizes, the applicable moisture content and dimensions used in design shall be noted in the plans and specifications.

TABLE 13.2.1A Net Dry Dimensions for Dressed Lumber

Nominal Thickness	Dry Thickness	Nominal Width	Dry Width
Dimension Lumber (inches):			
2	1-1/2	2	1-1/2
2-1/2	2	3	2-1/2
3	2-1/2	4	3-1/2
3-1/2	3	5	4-1/2
4	3-1/2	6	5-1/2
4-1/2	4	8	7-1/4
		10	9-1/4
		12	11-1/4
		14	13-1/4
		16	15-1/4
Beams and Stringers and Posts and Timbers (inches):			
5 and greater	1/2 less than nominal	5 and greater	1/2 less than nominal

13.2.2 Glued Laminated Timber

13.2.2.1 General

Glued laminated timber shall comply with the requirements of AASHTO M 168 and shall be manufactured using wet-use adhesives.

13.2.2.2 Dimensions

13.2.2.2.1 Structural calculations for glued laminated timber shall be based on the net finished dimensions.

13.4.3 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load does not exceed 1/500 of the span.

13.4.4 For timber deck structures with timber girders or stringers of equal stiffness, and cross-bracing or diaphragms sufficient in depth and strength to ensure lateral distribution of loads, the deflection may be computed by considering all girders or stringers as acting together and having equal deflection. When the cross-bracing or diaphragms are not sufficient to laterally distribute loads, deflection shall be distributed as specified for moment.

13.4.5 For concrete decks on wood girders or stringers, the deflection shall be assumed to be resisted by all beams or stringers equally.

13.5 DESIGN VALUES

13.5.1 General

Stress and modulus of elasticity values used for design, referred to as allowable design values, shall be the tabulated values modified by all applicable adjustments required by this Section. The actual stress due to loading shall not exceed the allowable stress.

13.5.2 Tabulated Values for Sawn Lumber

13.5.2.1 Tabulated values for sawn lumber are given in Table 13.5.1A for visually graded lumber and Table 13.5.1B for mechanically graded lumber. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1991 Edition of the NDS® and represent a partial listing of available species and grades. Refer to the 1991 Edition of the NDS® for a more complete listing.

13.5.2.2 Stress Grades in Flexure

13.5.2.2.1 The tabulated unit bending stress for Dimension (2 to 4 inches thick) and Post and Timber grades applies to material with the load applied either to the narrow or wide face.

13.5.2.2.2 The tabulated unit bending stress for Decking grades applies only when the load is applied to the wide face.

13.5.2.2.3 The tabulated unit bending stress for Beam and Stringer grades applies only when the load is applied to the narrow face. When Post and Timber sizes are graded to Beam and Stringer grade requirements, the tabulated unit bending stress for the applicable Beam and Stringer grades may be used.

13.5.2.2.4 Beam and Stringer grades are normally graded for use as a single, simple span. When used as a continuous beam, the grading provisions customarily applied to the middle third of the simple span length shall be applied to the middle two-thirds of the length for two-span beams, and to the entire length for beams continuous over three or more spans.

13.5.3 Tabulated Values for Glued Laminated Timber

13.5.3.1 Tabulated values for glued laminated timber of softwood species are given in Tables 13.5.3A and 13.5.3B. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1993 Edition of the American Institute of Timber Construction, AITC 117-93 Design, "Standard Specifications for Structural Glued Laminated Timber of Softwood Species." Refer to AITC 117-93 Design for a more complete listing.

13.5.3.2 Tabulated values for hardwood species shall be as given in the 1985 Edition of American Institute of Timber Construction, AITC 119, "Standard Specifications for Hardwood Glued Laminated Timber."

13.5.3.3 Species other than those specifically included or referenced in this Section may be used, provided that tabulated values are established for each species in accordance with AASHTO M 168.

13.5.4 Tabulated Values for Structural Composite Lumber

13.5.4.1 Representative tabulated design values for structural composite lumber are given in Table 13.5.4A for laminated veneer lumber and Table 13.5.4B for parallel strand lumber.

13.5.5 Adjustments to Tabulated Design Values

13.5.5.1 Wet Service Factor, C_M

13.5.5.1.1 Tabulated values for sawn lumber assume that the material is installed and used under continuously dry conditions where the moisture content of the wood does not exceed 19 percent. When the moisture content at installation or in service is expected to exceed 19 percent, tabulated values shall be reduced by the wet service fac-

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi)						Grading Rules Agency
		Bending F_b	Tension Parallel to Grain F_t	Shear Parallel to Grain F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity E	
DOUGLAS FIR-LARCH								
Select Structural		1450	1000	95	625	1700	1,900,000	
No. 1 & Btr	2″–4″ thick	1150	775	95	625	1500	1,800,000	WWPA
No. 1		1000	675	95	625	1450	1,700,000	
No. 2	2″ & wider	875	575	95	625	1300	1,600,000	WCLIB
Dense Select Structural		1900	1100	85	730	1300	1,700,000	
Select Structural	Beams and	1600	950	85	625	1100	1,600,000	
Dense No. 1	Stringers	1550	775	85	730	1100	1,700,000	
No. 1		1350	675	85	625	925	1,600,000	
No. 2		875	425	85	625	600	1,300,000	WCLIB
Dense Select Structural		1750	1150	85	730	1350	1,700,000	
Select Structural	Posts and	1500	1000	85	625	1150	1,600,000	
Dense No. 1	Timbers	1400	950	85	730	1200	1,700,000	
No. 1		1200	825	85	625	1000	1,600,000	
No. 2		750	475	85	625	700	1,300,000	
Dense Select Structural		1850	1100	85	730	1300	1,700,000	
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No. 1	Beams and	1550	775	85	730	1100	1,700,000	
No. 1	Stringers	1350	675	85	625	925	1,600,000	
Dense No. 2		1000	500	85	730	700	1,400,000	
No. 2		875	425	85	625	600	1,300,000	WWPA
Dense Select Structural		1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No. 1	Posts and	1400	950	85	730	1200	1,700,000	
No. 1	Timbers	1200	825	85	625	1000	1,600,000	
Dense No. 2		800	550	85	730	550	1,400,000	
No. 2		700	475	85	625	475	1,300,000	
EASTERN SOFTWOODS								
Select Structural		1250	575	70	335	1200	1,200,000	NELMA
No. 1	2″–4″ thick	775	350	70	335	1000	1,100,000	
No. 2	2″ & wider	575	275	70	335	825	1,100,000	NSLB

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi)						Grading Rules Agency
		Bending F_b	Tension Parallel to Grain F_t	Shear Parallel to Grain F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity E	
RED OAK								
Select Structural		1150	675	85	820	1000	1,400,000	
No. 1	2″–4″ thick	825	500	85	820	825	1,300,000	
No. 2	2″ & wider	800	475	85	820	625	1,200,000	
Select Structural	Beams and	1350	800	80	820	825	1,200,000	
No. 1	Stringers	1150	550	80	820	700	1,200,000	
No. 2		725	375	80	820	450	1,000,000	NELMA
Select Structural	Posts and	1250	850	80	820	875	1,200,000	
No. 1	Timbers	1000	675	80	820	775	1,200,000	
No. 2		575	400	80	820	350	1,000,000	
SOUTHERN PINE								
Select Structural		2850	1600	100	565	2100	1,800,000	SPIB
No. 1	2″–4″ thick	1850	1050	100	565	1850	1,700,000	
No. 2	2″–4″ wide	1500	825	90	565	1650	1,600,000	
Select Structural		2550	1400	90	565	2000	1,800,000	
No. 1	2″–4″ thick	1650	900	90	565	1750	1,700,000	
No. 2	5″–6″ wide	1250	725	90	565	1600	1,600,000	
Select Structural		2300	1300	90	565	1900	1,800,000	
No. 1	2″–4″ thick	1500	825	90	565	1650	1,700,000	
No. 2	8″ wide	1200	650	90	565	1550	1,600,000	
Select Structural	2″–4″ thick	2050	1100	90	565	1850	1,800,000	
No. 1	10″ wide	1300	725	90	565	1600	1,700,000	
No. 2		1050	575	90	565	1500	1,600,000	
Select Structural		1900	1050	90	565	1800	1,800,000	
No. 1	2″–4″ thick	1250	675	90	565	1600	1,700,000	
No. 2	12″ wide	975	550	90	565	1450	1,600,000	

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

1. Design values are taken from the 1991 Edition of the NDS® and are for a 10-year load duration and dry service conditions. Refer to the 1991 NDS® for additional species and grades and for a summary of grading rules agencies and commercial species classifications.

2. Wet Service Factor, C_M. When dimension lumber, 2″ to 4″ thick is used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors:

WET SERVICE FACTORS, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E
0.85*	1.0	0.97	0.67	0.8**	0.9

* when $(F_b)(C_F) \leq 1{,}150$ psi, $C_M = 1.0$
** when $F_c \leq 750$ psi, $C_M = 1.0$

When timbers 5″ by 5″ and larger are used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors (for Southern Pine and Mixed Southern Pine, use tabulated values without further adjustment):

WET SERVICE FACTORS, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E
1.00	1.00	1.00	0.67	0.91	1.00

3. Size Factor, C_F. For all species other than Southern Pine and Mixed Southern Pine, tabulated bending, tension, and compression parallel to grain design values for dimension lumber 2″ to 4″ thick shall be multiplied by the following size factors:

SIZE FACTORS, C_F

Grades	Width	F_b Thickness 2″ & 3″	F_b Thickness 4″	F_t	F_c
Select Structural, No. 1 & Btr. No. 1, No. 2, No. 3	2″, 3″ & 4″	1.5	1.5	1.5	1.15
	5″	1.4	1.4	1.4	1.1
	6″	1.3	1.3	1.3	1.1
	8″	1.2	1.3	1.2	1.05
	10″	1.1	1.2	1.1	1.0
	12″	1.0	1.1	1.0	1.0
	14″ & wider	0.9	1.0	0.9	0.9

TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued)

For Southern Pine and Mixed Southern Pine dimension lumber, 2″ to 4″ thick, appropriate size adjustment factors have been incorporated in tabulated values, with the following exceptions:

For dimension lumber 4″ thick, 8″ and wider, tabulated bending design values shall be multiplied by the size factor, $C_F = 1.1$.

For dimension lumber wider than 12″, tabulated bending, tension, and compression parallel to grain design values for 12″ wide lumber shall be multiplied by the size factor, $C_F = 0.9$.

4. Flat Use Factor, C_{fu}. Bending design values are based on edgewise use (load applied to narrow face). When dimension lumber 2″ to 4″ thick is used flatwise (load applied to wide face), the bending design value shall be multiplied by the following flat use factors:

FLAT USE FACTORS, C_{fu}

Width	Thickness 2″ & 3″	Thickness 4″
2″ & 3″	1.0	...
4″	1.1	1.0
5″	1.1	1.05
6″	1.15	1.05
8″	1.15	1.05
10″ & wider	1.2	1.1

5. Repetitive Member Factor, C_r. Bending design values for dimension lumber 2″ to 4″ thick shall be multiplied by the repetitive member factor $C_r = 1.15$, when such members are used as stringers, decking or similar members which are in contact or are spaced not more than 24″ on centers, are not less than 3 in number and are joined by load distributing elements adequate to support the design load.

6. Shear Stress Factor, C_H. Tabulated shear design values parallel to grain, F_V, have been reduced to allow for the occurrence of splits, checks, and shakes and may be multiplied by the shear stress factors given below when the length of split, or size of check or shake is known and no increase in them is anticipated. When the shear stress factor is applied to Southern Pine or Mixed Southern Pine, a tabulated design value of $F_V = 90$ lb/in.^{2} shall be used for all grades. Shear stress factors shall be linearly interpolated.

SHEAR STRESS FACTORS, C_H

Length of split on wide face of 2″ (nominal) lumber	C_H	Length of split on wide face of 3″ (nominal) and thicker lumber	C_H	Size of shake* in 2″ (nominal) and thicker lumber	C_H
no split	2.00	no split	2.00	no shake	2.00
1/2 × wide face	1.67	1/2 × narrow face	1.67	1/6 × narrow face	1.67
3/4 × wide face	1.50	3/4 × narrow face	1.50	1/4 × narrow face	1.50
1 × wide face	1.33	1 × narrow face	1.33	1/3 × narrow face	1.33
1-1/2 × wide face or more	1.00	1-1/2 × narrow face or more	1.00	1/2 × narrow face or more	1.00

* Shake is measured at the end between lines enclosing the shake and perpendicular to the loaded face.

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TABLE 13.5.1B Tabulated Design Values for Mechanically Graded Dimension Lumber

Species and Commercial Grade	Size Classification	Design Values in Pounds per Square Inch (psi) Bending F_b	Tension Parallel to Grain F_t	Compression Parallel to Grain F_c	Modulus of Elasticity E	Grading Rules Agency
MACHINE STRESS RATED (MSR) LUMBER						
900f-1.0E		900	350	1050	1,000,000	WCLIB, WWPA
1200f-1.2E		1200	600	1400	1,200,000	NLGA, SPIB, WCLIB, WWPA
1350f-1.3E		1350	750	1600	1,300,000	SPIB, WCLIB, WWPA
1450f-1.3E		1450	800	1625	1,300,000	NLGA, WCLIB, WWPA
1500f-1.3E		1500	900	1650	1,300,000	SPIB
1500f-1.4E		1500	900	1650	1,400,000	NLGA, SPIB, WCLIB, WWPA
1650f-1.4E		1650	1020	1700	1,400,000	SPIB
1650f-1.5E		1650	1020	1700	1,500,000	NLGA, SPIB, WCLIB, WWPA
1800f-1.6E		1800	1175	1750	1,600,000	NLGA, SPIB, WCLIB, WWPA
1950f-1.5E	2" & less in thickness	1950	1375	1800	1,500,000	SPIB
1950f-1.7E		1950	1375	1800	1,700,000	NLGA, SPIB, WWPA
2100f-1.8E	2" & wider	2100	1575	1875	1,800,000	NLGA, SPIB, WCLIB, WWPA
2250f-1.6E		2250	1750	1925	1,600,000	SPIB
2250f-1.9E		2250	1750	1925	1,900,000	NLGA, SPIB, WWPA
2400f-1.7E		2400	1925	1975	1,700,000	SPIB
2400f-2.0E		2400	1925	1975	2,000,000	NLGA, SPIB, WCLIB, WWPA
2550f-2.1E		2550	2060	2025	2,100,000	NLGA, SPIB, WWPA
2700f-2.2E		2700	2150	2100	2,200,000	NLGA, SPIB, WCLIB, WWPA
2850f-2.3E		2850	2300	2150	2,300,000	SPIB, WWPA
3000f-2.4E		3000	2400	2200	2,400,000	NLGA, SPIB
3150f-2.5E		3150	2500	2250	2,500,000	SPIB
3300f-2.6E		3300	2650	2325	2,600,000	SPIB
900f-1.2E		900	350	1050	1,200,000	NLGA, WCLIB
1200f-1.5E	2" & less in thickness	1200	600	1400	1,500,000	NLGA, WCLIB
1350f-1.8E		1350	750	1600	1,800,000	NLGA
1500f-1.8E	6" & wider	1500	900	1650	1,800,000	WCLIB
1800f-2.1E		1800	1175	1750	2,100,000	NLGA, WCLIB

1. Design values are taken from the 1991 Edition of the NDS® and are for a 10-year load duration and dry service conditions. Refer to the 1991 NDS® for additional grades and for a summary of grading rules agencies.
2. Design values for shear parallel to grain and compression perpendicular to grain shall be as specified in Table 13.5.1A for No. 2 visually graded dimension lumber of the appropriate species.
3. Use of the wet service factor, shear stress factor, repetitive member factor, and flat use factor shall be as specified in Table 13.5.1A for visually graded dimension lumber.

tors, C_M, given in footnotes to Tables 13.5.1A and 13.5.1B.

13.5.5.1.2 Tabulated values for glued laminated timber and structural composite lumber assume that the material is used under continuously dry conditions where the moisture content in service does not exceed 16 percent. When the moisture content in service is expected to exceed 16 percent, tabulated values shall be reduced by the wet service factors, C_M, given in the footnotes to Tables 13.5.3A and 13.5.3B for glued laminated timber and Tables 13.5.4A and 13.5.4B for structural composite lumber.

13.5.5.1.3 The moisture content of wood used in exposed bridge applications will normally exceed 19 percent and tabulated values shall be reduced by the wet service factor unless an analysis of regional, geographic, and climatological conditions that affect moisture content indicate that the in-service moisture content will not exceed 19 percent for sawn lumber and 16 percent for glued laminated timber and structural composite lumber over the life of the structure.

TABLE 13.5.2A Tabulated Design Values for Bearing Parallel to Grain

		Dry Service Conditions		
		Sawn Lumber		
Species Combination	Wet Service Conditions	5″ × 5″ & Larger	2″ to 4″ Thick	Glued Laminated Timber
Douglas Fir-Larch (Dense)	1570	1730	2360	2750
Douglas Fir-Larch	1350	1480	2020	2360
Eastern Softwoods	880	—	1340	—
Hem-Fir	1110	1220	1670	1940
Mixed Southern Pine	1270	1390	1900	—
Northern Red Oak	1150	1270	1730	2010
Red Maple	1100	1210	1650	1930
Red Oak	1010	1110	1520	1770
Southern Pine	1320	1450	1970	2300
Southern Pine (Dense)	1540	1690	2310	2690
Spruce-Pine-Fir	940	1040	1410	1650
Spruce-Pine-Fir (South)	810	900	1220	1430
Yellow Poplar	890	—	1340	1560

1. Design values are taken from the 1991 Edition of the NDS®. Refer to the 1991 NDS® for additional species.
2. Wet and dry service conditions are as defined in Article 13.5.5.1. The wet service factor has been applied to values tabulated for wet service conditions and further adjustment by this factor is not required.

13.5.5.2 Load Duration Factor, C_D

13.5.5.2.1 Wood can sustain substantially greater maximum loads for short load durations than for long load durations. Tabulated stresses for sawn lumber, glued laminated timber, and structural composite lumber are based on a normal load duration which contemplates that the member is stressed to the maximum stress level, either continuously or cumulatively, for a period of approximately 10 years, and/or stressed to 90 percent of the maximum design level continuously for the remainder of the member life.

13.5.5.2.2 When the full maximum load is applied either cumulatively or continuously for periods other than 10 years, tabulated stresses shall be multiplied by the load duration factor, C_D, given in Table 13.5.5A.

13.5.5.2.3 The provisions of this article do not apply to modulus of elasticity or to compression perpendicular to grain, but do apply to mechanical fastenings, except as otherwise noted. The load duration factor for impact does not apply to members pressure-impregnated with preservative salts to the heavy retentions required for marine exposure.

13.5.5.2.4 Increases in tabulated stresses resulting from various load duration factors are not cumulative and the load duration factor for the shortest duration load in a combination of loads shall apply for that load combination. The resulting structural members shall not be smaller than required for a longer duration of loading (refer to the 1991 Edition of the NDS® for additional commentary).

13.5.5.2.5 Modification of design stresses for load combinations, as specified in Section 3, are cumulative with load duration adjustments.

13.5.5.3 Adjustment for Preservative Treatment

Tabulated values apply to untreated wood and to wood that is preservatively treated in accordance with the requirements of AASHTO M 133. Unless otherwise noted, no adjustment of tabulated values is required for preservative treatment.

13.6 BENDING MEMBERS

13.6.1 General

13.6.1.1 The provisions of this article are applicable to straight members and to slightly curved bending members where the radius of curvature exceeds the span in inches divided by 800. Additional design requirements for

TABLE 13.5.5A Load Duration Factor, C_D

Load Duration	C_D
Permanent	0.90
2 months (vehicle live load)	1.15
7 days	1.25
1 day	1.33
5 minutes (railing only)	1.65

curved glued laminated timber members shall be as specified in the 1991 Edition of the NDS®.

13.6.1.2 For simple, continuous, and cantilevered bending members, the span shall be taken as the clear distance between supports plus one-half the required bearing length at each support.

13.6.1.3 Bending members shall be transversely braced to prevent lateral displacement and rotation and transmit lateral forces to the bearings. Transverse bracing shall be provided at the supports for all span lengths and at intermediate locations as required for lateral stability and load transfer (Article 13.6.4.4). The depth of transverse bracing shall not be less than ¾ the depth of the bending member.

13.6.1.4 Support attachments for bending members shall be of sufficient size and strength to transmit vertical, longitudinal and transverse loads from the superstructure to the substructure in accordance with the requirements of Section 3.

13.6.1.5 Glued laminated timber and structural composite lumber girders shall preferably be cambered a minimum 3 times the computed dead load deflection, but not less than ½ inch.

13.6.2 Notching

Notching of bending members can severely reduce member capacity and is not recommended. When notching is required for sawn lumber members, design limitations and requirements shall be in accordance with the NDS®, 1991 Edition. Design requirements and limitations for notching glued laminated timber members shall be as given in the "Timber Construction Manual," 1985 Edition by the American Institute of Timber Construction, published by John Wiley & Sons, New York, New York. Design requirements and limitations for notching structural composite lumber shall be as specified for glued laminated timber.

13.6.3 Modulus of Elasticity

The modulus of elasticity used for stiffness and stability computations shall be the tabulated modulus of elasticity adjusted by the applicable adjustment factor given in the following equation:

$$E' = EC_M \tag{13-1}$$

where:

E' = allowable modulus of elasticity in psi;
E = tabulated modulus of elasticity in psi;
C_M = wet service factor from Article 13.5.5.1.

13.6.4 Bending

13.6.4.1 Allowable Stress

The allowable unit stress in bending shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_b' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r \tag{13-2}$$

where:

F_b' = allowable unit stress in bending in psi
F_b = tabulated unit stress in bending in psi
C_M = wet service factor from Article 13.5.5.1
C_D = load duration factor from Article 13.5.5.2
C_F = bending size factor for sawn lumber and structural composite lumber, and for glued laminated timber with loads applied parallel to the wide face of the laminations, from Article 13.6.4.2
C_v = volume factor for glued laminated timber with loads applied perpendicular to the wide face of the laminations, from Article 13.6.4.3
C_L = beam stability factor from Article 13.6.4.4.
C_f = form factor from Article 13.6.4.5
C_{fu} = flat use factor for sawn lumber from footnotes to Tables 13.5.1A and 13.5.1B
C_r = repetitive member factor for sawn lumber from footnotes to Table 13.5.1A.

The volume factor, C_v, shall not be applied simultaneously with the beam stability factor, C_L, and the lesser of the two factors shall apply in Equation 13-2.

13.6.4.2 Size Factor, C_F

13.6.4.2.1 The tabulated bending stress, for dimension lumber 2 inches to 4 inches thick shall be multiplied by the bending size factor, C_F, given in the footnotes to Table 13.5.1A.

13.6.4.2.2 For rectangular sawn lumber bending members 5 inches or thicker and greater than 12 inches in depth, and for glued laminated timber with loads applied parallel to the wide face of the laminations and greater than 12 inches in depth, the tabulated bending stress shall be multiplied by the size factor, C_F, determined from the following relationship:

$$C_F = \left(\frac{12}{d}\right)^{1/9} \qquad (13\text{-}3)$$

where d is the member depth in inches.

13.6.4.2.3 For structural composite lumber bending members of any width, the tabulated bending stress shall be reduced by the size factor, C_F, given by the following equation:

$$C_F = (21/L)^{1/m}(12/d)^{1/m} \qquad (13\text{-}4)$$

where:

- L = length of bending member between points of zero moment in feet;
- d = depth of bending member in inches;
- m = parameter for the specific material determined in accordance with the requirements of ASTM D 5456.

13.6.4.3 Volume Factor, C_v

13.6.4.3.1 The tabulated bending stress for glued laminated timber bending members with loads applied perpendicular to the wide face of the laminations shall be adjusted by the volume factor, C_v, as determined by the following relationship:

$$C_V = (21/L)^{1/x}(12/d)^{1/x}(5.125/b)^{1/x} \leq 1.0 \qquad (13\text{-}5)$$

where:

- L = length of bending member between points of zero moment in feet;
- d = depth of bending member in inches;
- b = width of bending member in inches;
- x = 20 for Southern Pine;
- x = 10 for all other species.

13.6.4.3.2 When multiple piece width layups are used, the width of the bending member used in Equation 13-4 shall be the width of the widest piece used in the layup.

13.6.4.4 Beam Stability Factor, C_L

13.6.4.4.1 Tabulated bending values are applicable to members which are adequately braced. When members are not adequately braced, the tabulated bending stress shall be modified by the beam stability factor, C_L.

13.6.4.4.2 When the depth of a bending member does not exceed its width, or when lateral movement of the compression zone is prevented by continuous support and points of bearing have lateral support to prevent rotation, there is no danger of lateral buckling and $C_L = 1.0$. For other conditions, the beam stability factor shall be determined in accordance with the following provisions.

13.6.4.4.3 The bending member effective length, l_e, shall be determined from the following relationships for any loading condition:

$$\begin{array}{ll} l_e = 2.06 l_u & \text{when } l_u/d < 7 \\ l_e = 1.63 l_u + 3d & \text{when } 7 \leq l_u/d \leq 14.3 \\ l_e = 1.84 l_u & \text{when } l_u/d > 14.3 \end{array}$$

where:

- l_e = effective length in inches;
- l_u = unsupported length in inches;
- d = depth of bending member in inches.

If lateral support is provided to prevent rotation at the points of bearing, but no other lateral support is provided throughout the bending member length, the unsupported length, l_u, is the distance between points of bearing, or the length of a cantilever.

If lateral support is provided to prevent rotation and lateral displacement at intermediate points as well as at the bearings, the unsupported length, l_u, is the distance between such points of intermediate lateral support.

13.6.4.4.4 The slenderness ratio for bending members, R_B, is determined from the following equation:

$$R_B = \sqrt{\frac{l_e d}{b^2}} \leq 50 \qquad (13\text{-}6)$$

where:

- R_B = bending member slenderness ratio;
- d = depth of bending member in inches;
- b = width of bending member in inches.

13.6.4.4.5 The beam stability factor, C_L, shall be computed as follows on the next page.

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.90} - \sqrt{\frac{(1+F_{bE}/F_b^*)^2}{3.61} - \frac{F_{bE}/F_b^*}{0.95}} \quad (13\text{-}7)$$

$$F_{bE} = \frac{K_{bE}E'}{R_B^2} \quad (13\text{-}8)$$

where:

F_b^* = tabulated bending stress adjusted by all applicable adjustment factors given in Equation 13-2 except the volume factor, C_v, the beam stability factor, C_L, and the flat-use factor, C_{fu};

K_{bE} = 0.438 for visually graded sawn lumber 0.609 for glued laminated timber, structural composite lumber, and machine stress rated lumber;

E' = allowable modulus of elasticity in psi as determined by Article 13.6.3.

13.6.4.5 Form Factor, C_f

For bending members with circular cross sections the tabulated bending stress shall be adjusted by the form factor, C_f = 1.18. A tapered circular section shall be considered as a bending member of variable cross section.

13.6.5 Shear Parallel to Grain

13.6.5.1 General

13.6.5.1.1 The provisions of this article apply to shear parallel to grain (horizontal shear) at or near the points of vertical support of solid bending members. Refer to the 1991 edition of the NDS® for additional design requirements for other member types.

13.6.5.1.2 The critical shear in wood bending members is shear parallel to grain. It is unnecessary to verify the strength of bending members in shear perpendicular to grain.

13.6.5.2 Actual Stress

The actual unit stress in shear parallel to grain due to applied loading on rectangular members shall be determined by the following equation:

$$f_v = \frac{3V}{2bd} \quad (13\text{-}9)$$

where:

f_v = actual unit stress in shear parallel to grain in psi;
b = width of bending member in inches;
d = depth of bending member in inches;
V = vertical shear in pounds, as determined in accordance with the following provisions.

For uniformly distributed loads, such as dead load, the magnitude of vertical shear used in Equation 13-9 shall be the maximum shear occurring at a distance from the support equal to the bending member depth, d. When members are supported by full bearing on one surface, with loads applied to the opposite surface, all loads within a distance from the supports equal to the bending member depth shall be neglected.

For vehicle live loads, the loads shall be placed to produce the maximum vertical shear at a distance from the support equal to three times the bending member depth, 3d, or at the span quarter point, L/4, whichever is the lesser distance from the support. The distributed live load shear used in Equation 13-9 shall be determined by the following expression:

$$V_{LL} = 0.50\,[(0.60\,V_{LU}) + V_{LD}] \quad (13\text{-}10)$$

where:

V_{LL} = distributed live load vertical shear in pounds;
V_{LU} = maximum vertical shear, in pounds, at 3d or L/4 due to undistributed wheel loads;
V_{LD} = maximum vertical shear, in pounds, at 3d or L/4 due to wheel loads distributed laterally as specified for moment in Article 3.23.

For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member.

13.6.5.3 Allowable Stress

The allowable unit stress in shear parallel to grain shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_v' = F_v C_M C_D \quad (13\text{-}11)$$

where:

F_v' = allowable unit stress in shear parallel to grain in psi;
F_v = tabulated unit stress in shear parallel to grain in psi;
C_M = wet service factor from Article 13.5.5.1;
C_D = load duration factor from Article 13.5.5.2.

For sawn lumber beams, further adjustment by the shear stress factor may be applicable as described in the footnotes to Table 13.5.1A.

For structural composite lumber, more restrictive adjustments to the tabulated shear stress parallel to grain shall be as recommended by the material manufacturer.

13.6.6 Compression Perpendicular to Grain

13.6.6.1 General

When calculating the bearing stress in compression perpendicular to grain at beam ends, a uniform stress distribution shall be assumed.

13.6.6.2 Allowable Stress

The allowable unit stress in compression perpen-337dicular to grain shall be the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F'_{c\perp} = F_{c\perp} C_M C_b \tag{13-12}$$

where:

$F'_{c\perp}$ = allowable unit stress in compression perpendicular to grain, in psi;.
$F_{c\perp}$ = tabulated unit stress in compression perpendicular to grain, in psi;
C_M = wet service factor from Article 13.5.5.1;
C_b = bearing area factor from Article 13.6.6.3.

13.6.6.3 Bearing Area Factor, C_b

Tabulated values in compression perpendicular to grain apply to bearings of any length at beam ends, and to all bearings 6 inches or more in length at any other location. For bearings less than 6 inches in length and not nearer than 3 inches to the end of a member, the tabulated value shall be adjusted by the bearing area factor, C_b, given by the following equation:

$$C_b = \frac{l_b + 0.375}{l_b} \tag{13-13}$$

where l_b is the length of bearing in inches, measured parallel to the wood grain. For round washers, or other round bearing areas, the length of bearing shall be the diameter of the bearing area.

The multiplying factors for bearing lengths on small areas such as plates and washers are given in Table 13.6.1A.

TABLE 13.6.1A Values of the Bearing Area Factor, C_b, for Small Bearing Areas

Length of Bearing, l_b (in.)	1/2	1	1-1/2	2	3	4	6 or more
Bearing Area Factor, C_b	1.75	1.38	1.25	1.19	1.13	1.10	1.00

13.6.7 Bearing on Inclined Surfaces

For bearing on an inclined surface, the allowable unit stress in bearing shall be as given by the following equation:

$$F'_\theta = \frac{F'_g F'_{c\perp}}{F'_g \sin^2\theta + F'_{c\perp}\cos^2\theta} \tag{13-14}$$

where:

F'_θ = allowable unit stress for bearing on an inclined surface, in psi;
F'_g = allowable unit stress in bearing parallel to grain from Article 13.7.4;
$F_{c\perp}$ = allowable unit stress in compression perpendicular to the grain from Article 13.6.6;
θ = angle in degrees between the direction of load and the direction of grain.

13.7 COMPRESSION MEMBERS

13.7.1 General

13.7.1.1 The provisions of this article apply to simple solid columns consisting of a single piece of sawn lumber, piling, structural composite lumber, or glued laminated timber. Refer to the 1991 Edition of the NDS® for design requirements for built-up columns, consisting of a number of solid members joined together with mechanical fasteners, and for spaced columns consisting of two or more individual members with their longitudinal axes parallel, separated and fastened at the ends and at one or more interior points by blocking.

13.7.1.2 The term "column" refers to all types of compression members, including members forming part of a truss or other structural components.

13.7.1.3 Column bracing shall be provided where necessary to provide lateral stability and resist wind or other lateral forces.

13.7.2 Eccentric Loading or Combined Stresses

Members with eccentric loading or combined stresses shall be designed in accordance with the provisions of the NDS®, 1991 Edition.

13.7.3 Compression

13.7.3.1 Net Section

The actual unit stress in compression parallel to grain, f_c, shall be based on the net section as described in Article 13.1, except that it may be based on the gross section when the reduced section does not occur in the critical part of the column length that is most subject to potential buckling.

13.7.3.2 Allowable Stress

The allowable unit stress in compression parallel to grain shall not exceed the tabulated stress adjusted by the applicable adjustment factors given in the following equation:

$$F_c' = F_c C_M C_D C_F C_P \tag{13-15}$$

where:

- F_c' = allowable unit stress in compression parallel to grain in psi;
- F_c = tabulated unit stress in compression parallel to grain in psi;
- C_M = wet service factor from Article 13.5.5.1;
- C_D = load duration factor from Article 13.5.5.2;
- C_F = compression size factor for sawn lumber from footnotes to Table 13.5.1A;
- C_P = column stability factor from Article 13.7.3.3.

13.7.3.3 Column Stability Factor, C_P

13.7.3.3.1 Tabulated values in compression parallel to grain are applicable to members which are adequately braced. When members are not adequately braced, the tabulated stress shall be modified by the column stability factor, C_P.

13.7.3.3.2 When a compression member is supported throughout its length to prevent lateral displacement in all directions, $C_P = 1.0$. For other conditions, the column stability factor shall be determined in accordance with the following provisions.

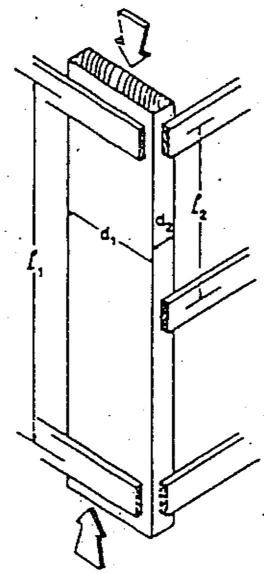
13.7.3.3.3 The effective column length, l_e, shall be determined in accordance with good engineering practice. Actual column length, l, may be multiplied by an effective length factor to determine the effective column length:

$$l_e = Kl \tag{13-16}$$

where:

- l_e = effective column length in inches
- K = effective length factor from Table C-1 of Appendix C
- l = actual column length between points of lateral support in inches.

13.7.3.3.4 For columns of rectangular cross section, the column slenderness ratio, l_e/d, shall be taken as the larger of the ratios, l_{e1}/d_1 or l_{e2}/d_2. (See Figure 13.7.1A.) The slenderness ratio shall not exceed 50.



l_1 and l_2 = distances between points of lateral support in planes 1 and 2, inches.
d_1 and d_2 = cross-sectional dimensions of rectangular compression member in planes of lateral support, inches.

FIGURE 13.7.1A

13.7.3.3.5 The column stability factor, C_P, shall be as given by the following expressions:

$$C_P = \frac{1 + F_{cE}/F_c^*}{2c} - \sqrt{\frac{\left(1 + F_{cE}/F_c^*\right)^2}{(2c)^2} - \frac{F_{cE}/F_c^*}{c}} \tag{13-17}$$

$$F_{cE} = \frac{K_{cE}E'}{(l_e/d)^2} \tag{13-18}$$

TABLE 13.7.1A Support Condition Coefficients for Tapered Columns

Support Condition	Support Condition Coefficient, a
Large end fixed, small end unsupported	0.70
Small end fixed, large end unsupported	0.30
Both ends simply supported	
Tapered toward one end	0.50
Tapered towards both ends	0.70

where:

F_c^* = tabulated stress in compression parallel to grain adjusted by all applicable modification factors given in Equation 13-14 except C_P;
K_{cE} = 0.300 for visually graded sawn lumber; 0.418 for glued laminated timber, structural composite lumber, and machine stress-rated lumber;
c = 0.80 for sawn lumber;
0.85 for round piles;
0.90 for glued laminated timber and structural composite lumber.

For especially severe service conditions or extraordinary hazardous conditions, the use of lower design values than those obtained above may be necessary. Refer to the 1991 Edition of the NDS®.

13.7.3.4 Tapered Columns

13.7.3.4.1 For rectangular columns tapered at one or both ends, the cross-sectional area shall be based on the representative dimension of each tapered face. The representative dimension, d_{rep}, of each tapered face shall be based on the support condition coefficient given in Table 13.7.1A.

13.7.3.4.2 For support conditions given in Table 13.7.1A, the representative dimension, d_{rep}, of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + (d_{max} - d_{min})\left[a - 0.15\left(1 - \frac{d_{min}}{d_{max}}\right)\right] \qquad (13\text{-}19)$$

where:

d_{rep} = representative dimension for a tapered column face, in inches;
d_{min} = minimum column face dimension, in inches;
d_{max} = maximum column face dimension, in inches;
a = coefficient based on support conditions.

13.7.3.4.3 For support conditions other than those in Table 13.7.1A, the representative dimension of each tapered face shall be as given by the following equation:

$$d_{rep} = d_{min} + 0.33(d_{max} - d_{min}) \qquad (13\text{-}20)$$

13.7.3.4.4 For any tapered column, the actual stress in compression parallel to grain, f_c, shall not exceed the allowable stress determined by Equation 13-14, assuming the column stability factor $C_P = 1.0$.

13.7.3.5 Round Columns

The design of a round column shall be based on the design of a square column of the same cross-sectional area with the same degree of taper.

13.7.4 Bearing Parallel to Grain

13.7.4.1 The actual stress in bearing parallel to grain shall be based on the net area and shall not exceed the tabulated stress for bearing parallel to grain adjusted by the applicable adjustment factor given in the following equation:

$$F'_g = F_g C_D \qquad (13\text{-}21)$$

where:

F'_g = allowable unit stress in bearing parallel to grain in psi;
F_g = tabulated unit stress in bearing parallel to grain from Table 13.5.2A, in psi;
C_D = load duration factor from Article 13.5.5.2.

13.7.4.2 When the bearing load is at an angle to the grain, the allowable bearing stress shall be determined by Equation 13-14, using the design values for end-grain bearing parallel to grain and design values in compression perpendicular to grain.

13.7.4.3 When bearing parallel to grain exceeds 75% of the allowable value determined by Equation 13-21, bearing shall be on a metal plate or on other durable, rigid, homogeneous material of adequate strength and stiffness to distribute applied loads over the entire bearing area.

13.8 TENSION MEMBERS

13.8.1 Tension Parallel to Grain

The allowable unit stress in tension parallel to grain hall be the tabulated value adjusted by the applicable adjustment factors given in the following equation:

$$F'_t = F_t C_M C_D C_F \qquad (13\text{-}22)$$

where:

F_t' = allowable unit stress in tension parallel to grain in psi;
F_t = tabulated unit stress in tension parallel to grain in psi;
C_M = wet service factor from Article 13.5.5.1;
C_D = load duration factor from Article 13.5.5.2;
C_F = tension size factor for sawn lumber from footnotes to Table 13.5.1A and for structural composite lumber from footnotes to Tables 13.5.4A and 13.5.4B.

13.8.2 Tension Perpendicular to Grain

Designs which induce tension perpendicular to the grain of wood members should not be used. When tension perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist all such forces should be used. Refer to the 1991 Edition of the NDS® for additional information.

13.9 MECHANICAL CONNECTIONS

13.9.1 General

13.9.1.1 Except as otherwise required by this specification, mechanical connections and their installation shall conform to the requirements of the NDS®, 1991 Edition.

13.9.1.2 Components at mechanical connections, including the wood members, connecting elements, and fasteners, shall be proportioned so that the design strength equals or exceeds the required strength for the loads acting on the structure. The strength of the connected wood components shall be evaluated considering the net section, eccentricity, shear, tension perpendicular to grain and other factors that may reduce component strength.

13.9.2 Corrosion Protection

13.9.2.1 Except as permitted by this section, all steel hardware for wood structures shall be galvanized in accordance with AASHTO M 232 or cadmium plated in accordance with AASHTO M 299.

13.9.2.2 All steel components, timber connectors, and castings, other than malleable iron, shall be galvanized in accordance with AASHTO M 111.

13.9.2.3 Alternative corrosion protection coatings, such as epoxies, may be used when the demonstrated performance of the coating is sufficient to provide adequate protection for the intended exposure conduction.

13.9.2.4 Heat-treated alloy components and fastenings shall be protected by an approved alternative protective treatment that does not adversely affect the mechanical properties of the material.

13.9.3 Fasteners

13.9.3.1 Fastener design values shall be adjusted by the applicable adjustment factors for the intended use condition.

13.9.3.2 When determining fastener design values, wood shall be assumed to be used under wet-use or exposed to weather conditions.

13.9.3.3 Glulam rivets shall not be used in permanent structures.

13.9.4 Washers

13.9.4.1 Washers shall be provided under bolt and lag screw heads and under nuts that are in contact with wood. Washers may be omitted under heads of special timber bolts or dome-head bolts when the size and strength of the head is sufficient to develop connection strength without excessive wood crushing.

13.9.4.2 Washers shall be of sufficient size and strength to prevent excessive wood crushing when the fastener is tightened. For bolts or rods loaded in tension, washers shall be of sufficient size and strength to develop the tensile strength of the connection without excessive bending or exceeding wood strength in compression perpendicular to grain.

From: 2002
AASHTO STD SPECS

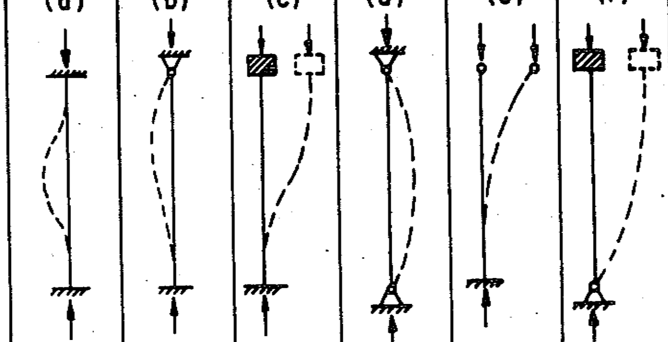
APPENDIX C *(Continued)*

EFFECTIVE LENGTH FACTOR, K

The Effective Length of a column, KL, has been used in the equations for allowable compression stress in the column. K is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. KL represents the length between inflection points of a buckled column. Restraint against rotation and translation of column ends influences the position of the inflection points in a column. Theoretical values of K for some idealized column end conditions are given in Table C-1. Since column end conditions seldom comply fully with idealized restraint against rotation and translation, the recommended values suggested by the Column Research Council are higher than the idealized values.

Columns in continuous frames unbraced by adequate attachment to shear walls, diagonal bracing, or adjacent structures depend on the bending stiffness of the rigidly connected beams for lateral stability. The effective length factor, K, is dependent on the amount of bending stiffness supplied by the beams at the column ends. If the amount of stiffness supplied by the beams is small, the value of K could exceed 2.0.

TABLE C-1

EFFECTIVE LENGTH FACTORS, K						
	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL K VALUE	0.5	0.7	1.0	1.0	2.0	2.0
DESIGN VALUE OF K WHEN IDEAL CONDITIONS ARE APPROXIMATED[1]	0.65	0.80	1.2	1.0	2.1	2.0
END CONDITION CODE		ROTATION FIXED TRANSLATION FIXED ROTATION FREE TRANSLATION FIXED ROTATION FIXED TRANSLATION FREE ROTATION FREE TRANSLATION FREE				

[1]For riveted and bolted truss members (partially restrained), use K = 0.75. For pinned connections in truss members, use K = 0.875 (pin friction).

If it is assumed that elastic action occurs and that all columns buckle simultaneously in a frame, it can be rationally shown that*

$$\frac{G_aG_b(\pi/K)^2 - 36}{6(G_a + G_b)} = \frac{\pi/K}{\tan(\pi/K)} \qquad \text{(C-1)}$$

where subscripts a and b refer to the two ends of the column.

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} \qquad \text{(C-2)}$$

Σ = summation of all members rigidly connected to an end of the column in the plane of bending;

I_c = moment of inertia of column;

L_c = unbraced length of column;

I_g = moment of inertia of beam or other restraining member;

L_g = unsupported length of beam or other restraining member;

K = effective length factor.

*See "Steel Structures Design and Behavior" by Charles G. Salmon and John E. Johnson, published by International Text Book Company, 1971.

Table C-2 is a graphical representation of the relationship between K, G_a, and G_b, and can be used to obtain the value of K easily. In frames which have columns that fall in the inelastic buckling range, (i.e., $KL/r < C_c = (2\pi^2\ E/F_y$ $1/2)$, K may often be reduced. The procedure for reducing K can be found in "Effective Length of Columns in Unbraced Frames" by Joseph A. Yura, *AISC Engineering Journal,* published by the American Institute of Steel Construction, 101 Park Avenue, New York, New York 10017.

From: Forest Service, Timber Bridge, 1992

vehicle. It is generally most convenient to start with a selected species and grade of lumber, size the deck thickness based on deflection, and then check bending and shear. Because of the susceptibility of the deck to loosening or delamination, a maximum live load deflection of $L/500$ is recommended. Effective deck section properties and typical dead loads are given in Tables 8-3 and 8-4, respectively.

Table 8-3.—Effective deck section properties for continuous longitudinal nail-laminated decks and longitudinal nail-laminated deck panels with adequate shear transfer between panels.

	H 15-44 and HS 15-44 12,000-pound wheel load				H 20-44 and HS 20-44 16,000-pound wheel load			
t (in.)	D_w (in.)	A (in²)	S (in³)	I (in⁴)	D_w (in.)	A (in²)	S (in³)	I (in⁴)
5-1/2	28.32	155.76	142.78	392.65	31.00	170.50	156.29	429.80
6	29.32	175.92	175.92	527.76	32.00	192.00	192.00	576.00
7-1/4	31.82	230.70	278.76	1,010.49	34.50	250.13	302.23	1,095.60
8	33.32	266.56	355.41	1,421.65	36.00	288.00	384.00	1,536.00
9-1/4	35.82	331.34	510.81	2,362.49	38.50	356.13	549.03	2,539.25
10	37.32	373.20	622.00	3,110.00	40.00	400.00	666.67	3,333.33
11-1/4	39.82	447.98	839.95	4,724.74	42.50	478.13	896.48	5,042.72
12	41.32	495.84	991.68	5,950.08	44.00	528.00	1,056.00	6,336.00
13-1/4	43.82	580.62	1,282.19	8,494.52	46.50	616.13	1,360.61	9,014.04
14	45.32	634.48	1,480.45	10,363.17	48.00	672.00	1,568.00	10,976.00
15-1/4	47.82	729.26	1,853.52	14,133.11	50.50	770.13	1,957.40	14,925.[illegible]
16	49.32	789.12	2,104.32	16,834.56	52.00	832.00	2,218.67	17,749.33

Table 8-4.—Deck dead load for the wheel distribution width (D_w) in lb/ft of deck span for longitudinal continuous nail-laminated decks and longitudinal nail-laminated deck panels with adequate shear transfer between panels.

	H 15-44 and HS 15-44 12,000-pound wheel load			H 20-44 and HS 20-44 16,000-pound wheel load		
t (in.)	D_w (in.)	Deck only (lb/ft)	Deck plus 3 in. of asphalt (lb/ft)	D_w (in.)	Deck only (lb/ft)	Deck plus 3 in. of asphalt (lb/ft)
5-1/2	28.32	54.08	142.58	31.00	59.20	156.08
6	29.32	61.08	152.71	32.00	66.66	166.67
7-1/4	31.82	80.10	179.54	34.50	86.84	194.65
8	33.32	92.55	196.67	36.00	99.99	212.50
9-1/4	35.82	115.04	226.97	38.50	123.65	243.97
10	37.32	129.58	246.21	40.00	138.88	263.88
11-1/4	39.82	155.54	279.97	42.50	166.01	298.83
12	41.32	172.16	301.28	44.00	183.32	320.82
13-1/4	43.82	201.59	338.53	46.50	213.92	359.23
14	45.32	220.29	361.95	48.00	233.32	383.32
15-1/4	47.82	253.20	402.66	50.50	267.39	425.20
16	49.32	273.98	428.11	52.00	288.87	451.40

From: Duluth Tech College & MN/DOT 1994

HS 15 Loading:

Use 3/4 of the values obtained from the formulas for HS 20 loading

Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading.

3.25.2 E. Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be considered a point loading.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: width of plank
Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected laminated floor, with adequate shear transfer between panels, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

3.25.3 F. Longitudinal Glued Laminated Timber Decks

3.25.3.1 **Bending Moment**

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

Longitudinal nail-laminated decks are constructed from lumber laminations that are 2 to 4 inches thick, and 5 inches or wider, in the Joist and Plank size classification.[11, 12] Both continuous and panelized configurations can be constructed from any lumber size provided it is a minimum c 6 inches in nominal depth (AASHTO 3.25.2.2). From a practical standpoint, however, continuous decks are normally constructed of 2-inch nominal material that is 6 to 12 inches wide to facilitate field nailing and handling. Panelized systems commonly use 4-inch nominal material that i 10 to 16 inches wide, which is more economical and practical for shop fabrication.

CONTINUOUS NAIL-LAMINATED LUMBER BRIDGES

Continuous nail-laminated lumber bridges are practical for simple spans up to approximately 19 feet for HS 20-44 and H 20-44 loads and 21 feet for HS 15-44 and H 15-44 loads. Load distribution and continuity across the bridge are provided by the nails that are placed through two and one-half laminations, in the same pattern used for transverse nail-laminated decks (Figure 8-10). Transverse stiffener beams are not required. The performance of longitudinal nail-laminated bridges is similar in many respects to transverse nail-laminated decks and depends primarily on the effectiveness of the nails in transferring loads between adjacent laminations. Field experience has shown that many nail-laminated decks demonstrate a tendency to loosen or delaminate from cyclic loading and moisture content changes in the laminations. This subsequently leads to reduced load distribution and deterioration of asphalt wearing surfaces. In longitudinal deck bridges, the potential for delamination is normally higher than for transverse configurations because the deck spans and associated deflections are generally larger. Performance can be improved by limiting live load deflections and using edge-grain lumber for laminations, but these measures may not be totally effective in eliminating deck loosening.

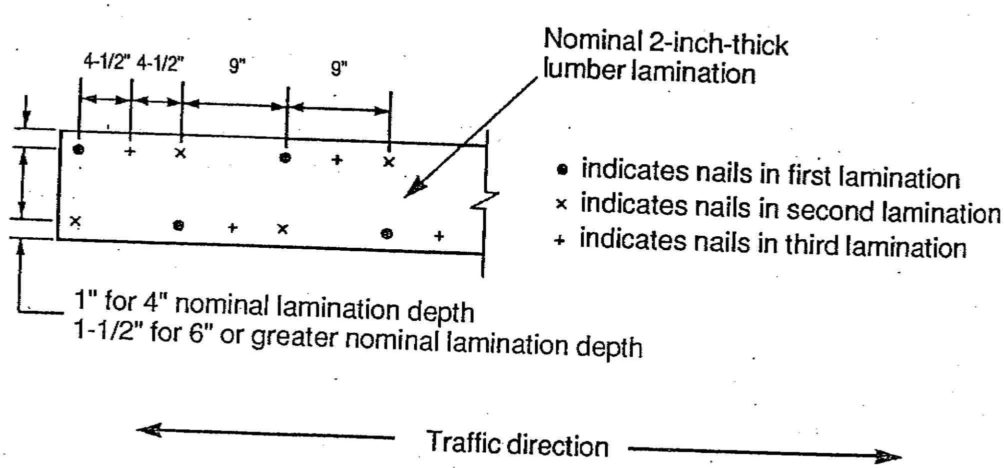


Figure 8-10.—Nailing pattern for continuous longitudinal nail-laminated lumber decks constructed of nominal 2-inch-thick sawn lumber.

From: US Forest Service, Timber Bridges Design, Construction, Inspection & Maintenance, 1992

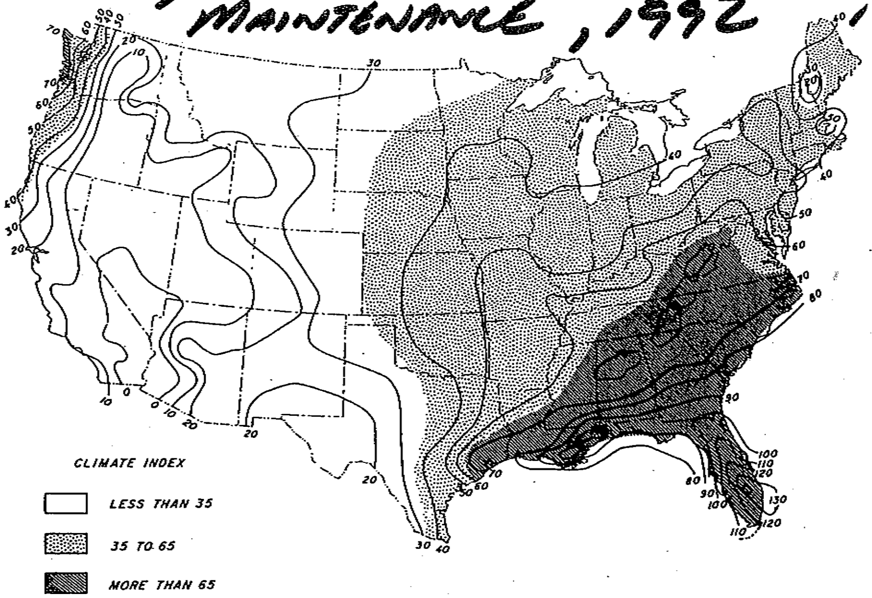


Figure 13-35.—Climate index map for decay hazard. The higher numbers indicate a higher decay hazard.

Areas Susceptible to Decay

Wood decay can occur only when proper conditions prevail for fungal growth. Although timber bridges differ in many respects, there are several common areas where decay is most likely to occur. These areas involve situations where the wood moisture is high and where breaks in the preservative envelope (or insufficient preservative penetration) provide an entry point for decay organisms. Signs of high moisture content and sites around fasteners, checks, or mechanical damage should be considered areas of high decay potential (Figure 13-36).

The moisture content of bridge components is not uniform, and substantial variations occur within and between members. End-grain surfaces absorb water much quicker than do side-grain surfaces (Figure 13-37). With other conditions equal, permeability in the longitudinal direction (parallel to grain) is 50 to 100 times greater than in the transverse direction (perpendicular to grain). Decay development is most affected by the moisture content of the wood in the immediate vicinity of the infection. Therefore, a member may remain generally dry and uninfected along most of its length but be severely decayed in localized areas where untreated wood is exposed and water is continuously or intermittently trapped. Bridge moisture conditions are also subject to seasonal variations and may be altered by maintenance operations or changes in drainage patterns. Wood that appears thoroughly dry may have been exposed to high moisture contents in the past and could be seriously decayed. The inspector must be alert for any visual or intuitive indications of wetting. Visual signs may appear as

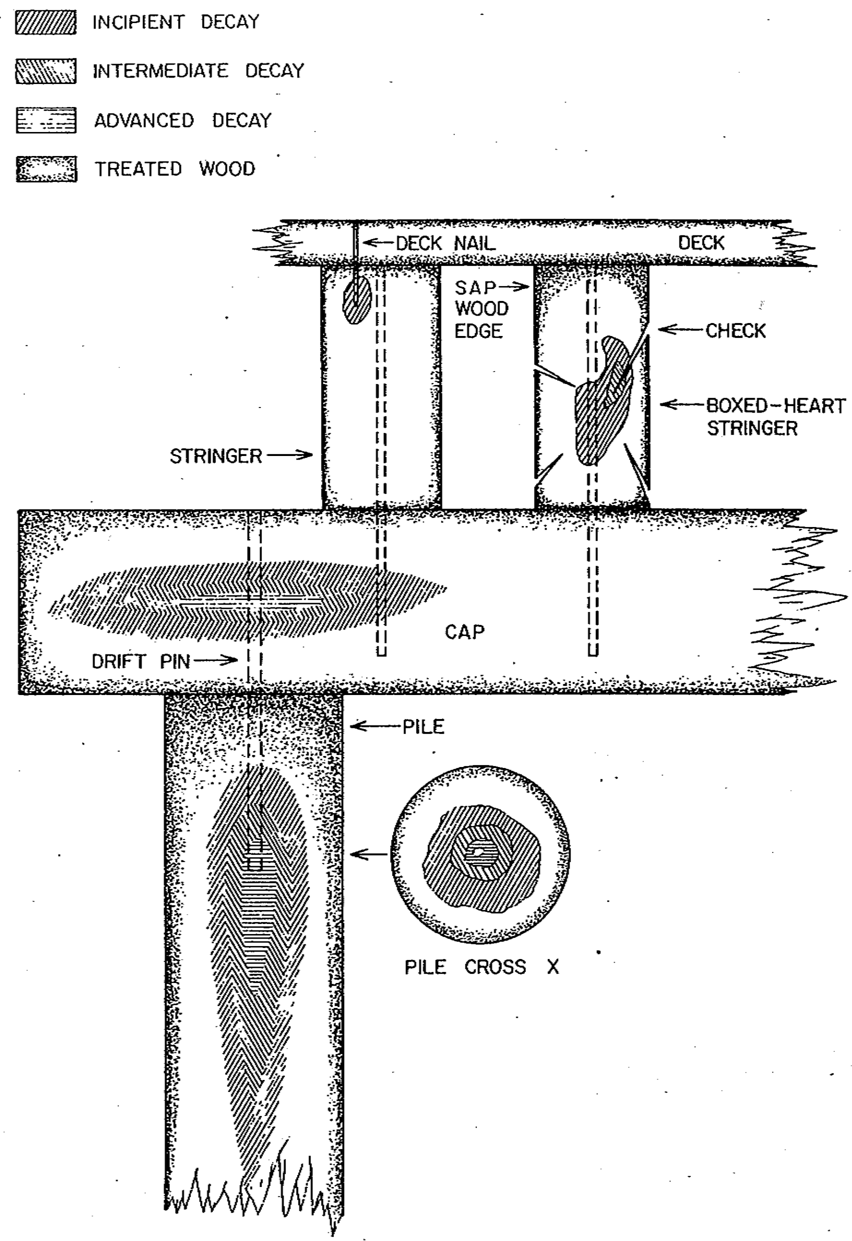


Figure 13-36.—Diagram depicting potential decay locations in a timber bridge.

watermarks, staining, or light mud stains. Intuitive signs include any horizontal surfaces, contact areas, depressions, or other features that may trap water and therefore indicate potentially high moisture exposures.

As discussed, the potential for bridge decay is highest where untreated wood is exposed. This condition occurs most often in the vicinity of seasoning checks, fasteners, and areas of mechanical damage. Conditions for deterioration are enhanced at these locations because moisture enters cracks or other crevices where air circulation and drying are inhibited. Seasoning checks commonly develop in large lumber members, and, t

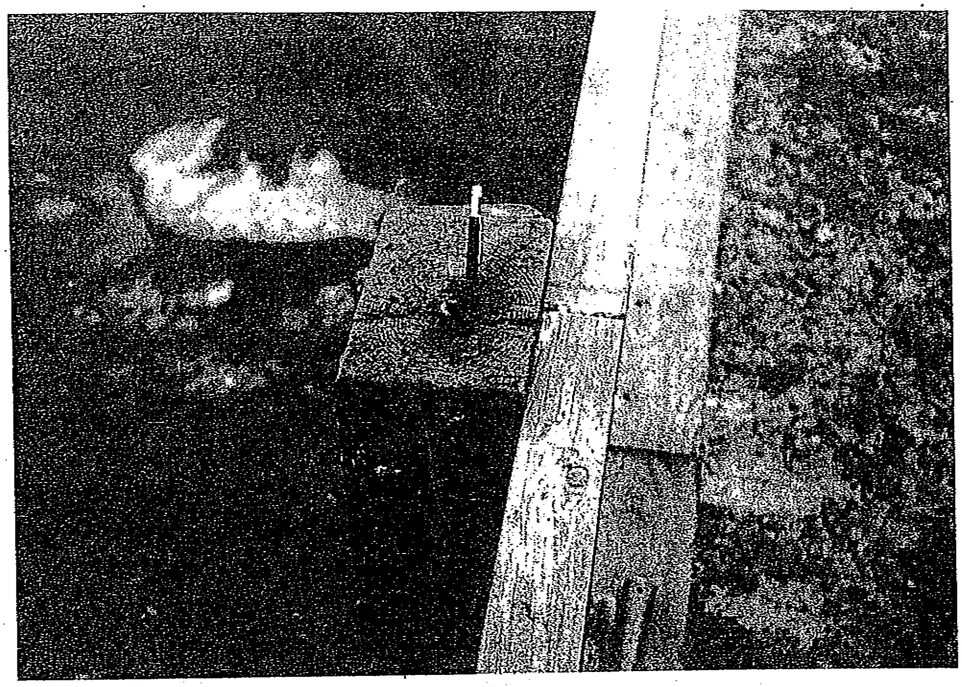


Figure 13-37.—Decay in the end grain of a timber rail post (photo courtesy of Duane Yager, USDA Forest Service).

lesser extent, in glulam. Although the size of the check influences the area of exposed untreated material, very small openings are still sufficient to allow entry of decay organisms (Figure 13-38). Holes for bolts, nails, or other hardware can trap water, which will be absorbed deep into the wood end grain by capillary action. Decay susceptibility at connections is higher because fasteners may be placed in field-bored holes that are not adequately treated with preservatives (Figure 13-39). Mechanical damage from improper handling, overloads, vehicle abrasion, and support settlements also breaks the preservative barrier and provides an entry point for decay organisms. In addition to increasing the decay hazard, mechanical damage may also affect structural capacity, depending on the decay's nature, location, and extent (Figure 13-40).

Component Inspection

Component inspection involves the systematic examination of individual bridge members. When deterioration is found, its location and extent must be defined and noted so that the load-carrying capacity of the structure can be determined by engineering analysis. At some locations, deterioration may have no significant effect on member strength. In other locations, any deterioration will reduce capacity. In both cases, the inspector must accurately locate, define, and record all deterioration, notwithstanding its perceived effects on structural capacity.

Because of the large number of structural components and the variety of locations where conditions for decay development exist in a bridge, the degree of accuracy for assessing the extent of deterioration depends on the judgment of the inspector. Regardless of bridge size, no inspection can

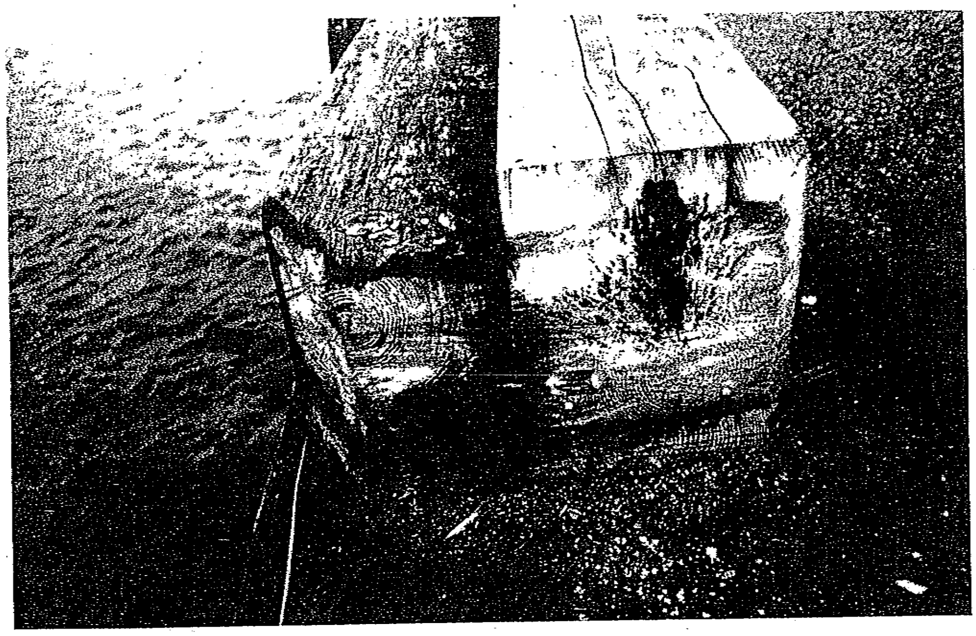


Figure 13-38.—Cross section of a timber curb, exposed by sawing, reveals interior decay resulting from seasoning checks in the upper surface.

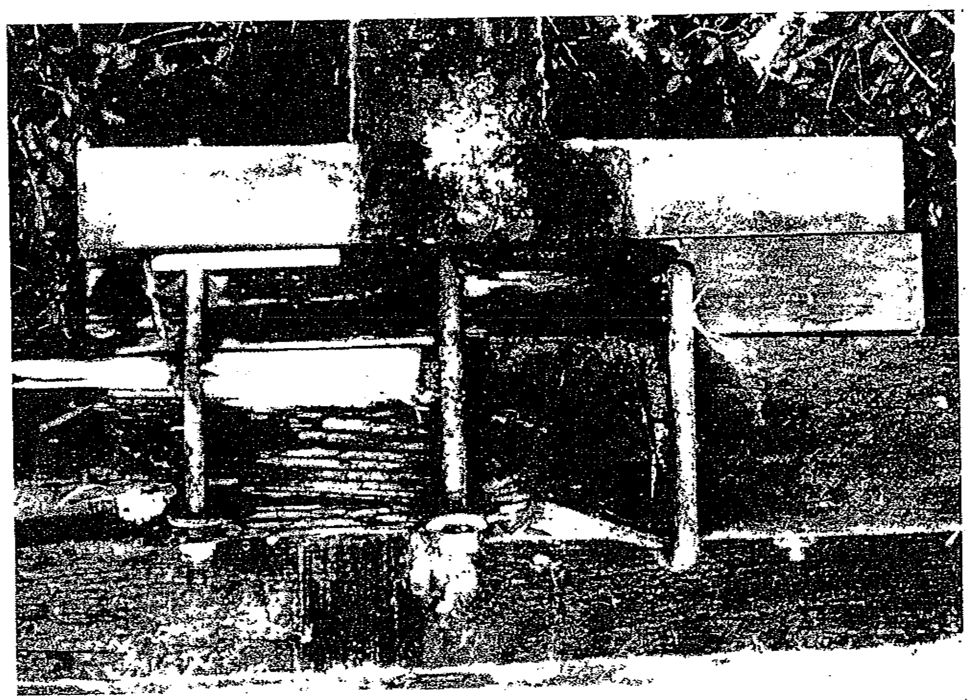


Figure 13-39.—Decay in timber members around field-bored fastener holes.

Figure 13-40.—Large crack in a sawn lumber bridge beam caused by vehicle overloads (photo courtesy of Duane Yager, USDA Forest Service).

reasonably or economically examine every bridge component. Rather, the inspector must base the degree of inspection on information from the preinspection evaluation and knowledge of bridge deterioration and its causes, signs, and probable locations. For example, it may not be practical to examine the area around each fastener when deck members are attached with penetrating fasteners in each beam. Instead, the inspector should select the most probable areas of deterioration for evaluation. If deterioration is found, its extent is determined and additional inspections are made at other locations. If no deterioration is found in high-hazard zones, it is unlikely that other areas are affected.

One of the most important aspects of component inspection is the sequence and coordination of inspection efforts. To ensure that all critical areas are covered, a systematic, well-defined plan must be developed. When more than one inspector is involved, the responsibilities of each must be clearly defined to avoid either missing areas or excessive duplication. The preferred inspection sequence generally follows the sequence of construction. After initially surveying the structure, the inspector begins with the lower substructure members and progresses upward to the top of the superstructure. Following this sequence, the inspector can observe the behavior of members under load before their actual inspection.

Initial Survey

The best way to begin a bridge inspection is to take a brief walk across and around the structure, observing general features and looking for obvious signs of deterioration or distress. Particular attention should be given to changes in the longitudinal or transverse deck elevation that may indicate foundation movement, deck swelling, or other adverse conditions. The rail and curb elements should also be checked for position and alignment. Slanted posts or separated rails may indicate deck swelling or superstructure movement. This is also a good time to observe drainage patterns on approach roadways and obstructions to deck drains, as well as the effectiveness of the deck and wearing surface in protecting underlying components. General observations of this type can alert the inspector to potentially adverse situations requiring more detailed examination later in the inspection. This inspection also can provide an opportunity to prepare initial sketches of the structure and to define the directions and other features used in recording inspection findings.

Substructure Inspection

The substructure is the portion of a bridge that is probably most susceptible to deterioration. Soil-contacting members such as posts, piling, abutments, and wing walls are exposed in varying degrees to nearly constant wetting, resulting in wood moisture contents suitable for decay. Surrounding soil frequently contains large numbers of fungal spores and woody plant material in which decay fungi can live and spread to infect bridge members. Substructure decay potential is also greater because of the high incidence of field fabrication (cutting and drilling) and the large number of penetrating fasteners.

Initial inspection of the substructure should begin with a visual examination of abutments for signs of deterioration, mechanical damage, and settlement. The most probable locations for decay are in the vicinity of the ground line, at connections between the cap and column, and at framing connections for bracing, tie rods, and backwall or wingwall planks. Starting at the base of the abutment, soil should be removed around a representative number of members in order to inspect for indications of decay or insect attack. When soil is very wet or covered by water, decay is generally limited to areas close to ground level because the lack of oxygen below the surface limits the growth of most fungi. As soil moisture content decreases, conditions below ground become more favorable, and decay may occur at depths of 2 feet or more in moderately dry soils. Surface decay and insect damage can be revealed by visual observation and probing. When evidence of decay is found, its extent is further defined by drilling or coring (Figure 13-41). Detecting internal decay is generally accomplished by using a combination of sounding and drilling or coring. Because sounding will reveal only serious internal defects, it should never be the only method used.

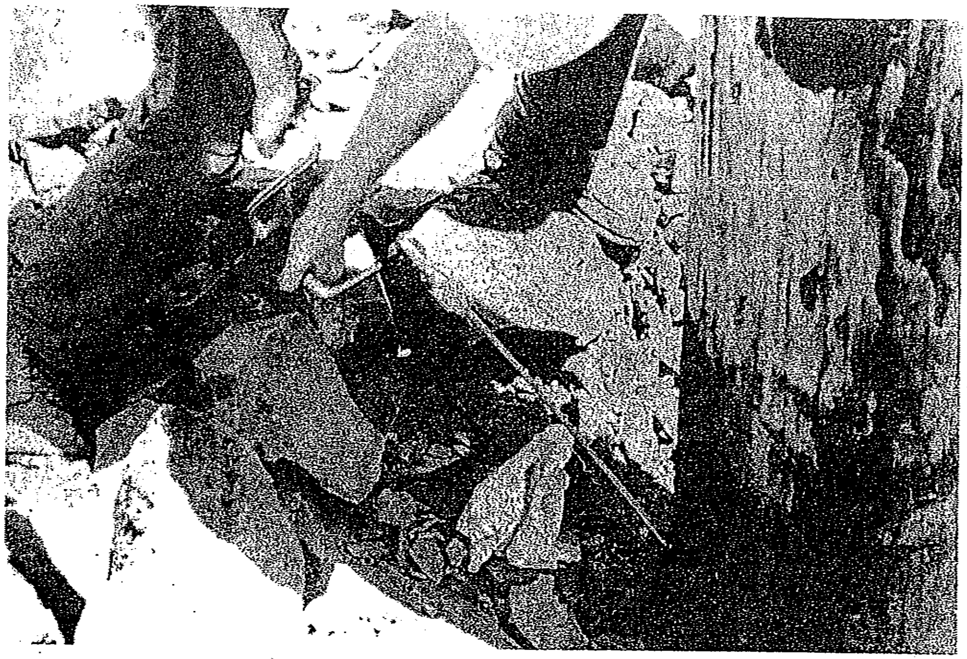


Figure 13-41.—Hand drilling at the base of a timber pile.

From below the ground line, inspection should proceed upward, with particular attention given to connections, seasoning checks, and mechanical damage. Timber backwalls, wingwalls, and incidental bracing should also be examined for breakage or bulging from earth pressure. Exposed end grain on pile or post tops should also be inspected for decay. Many tops are intentionally cut at an angle in the belief that water will run off. Instead, angled cuts expose more untreated end grain, increasing the decay potential. When tops are provided with protective sheet-metal caps, the condition of the cap should be checked for holes or tears in the surface. Damaged caps allow water to enter through the break and penetrate end grain, creating ideal conditions for internal decay (Figure 13-42).

Above the supporting piles or posts, the cap supporting the superstructure provides a horizontal surface that traps debris and water runoff from the deck. Connections into the cap and horizontal checks that trap water and debris are critical zones. The connection between the cap and column is especially important because many connections are made with drift pins or bolts that extend deep into the column end grain. Water from the cap flows into these connections and can result in substantial internal decay with little evidence of exterior damage (Figure 13-43). The inspector should also check for crushed zones at bearing points along the cap that trap water and damage the treated wood shell. Crushing can also indicate overloads or load redistribution from settlement and should be further investigated in other components of the structure.

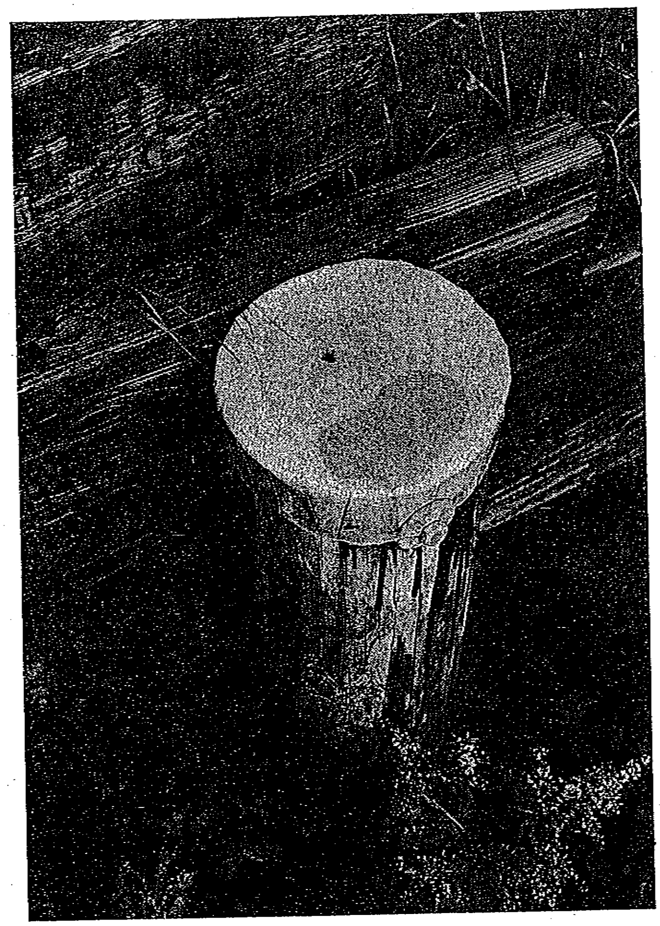


Figure 13-42.—(Top) Damaged metal pile caps allow water to enter, but restrict air circulation and drying. (Bottom) Pile decay is exposed when the damaged cap is removed.

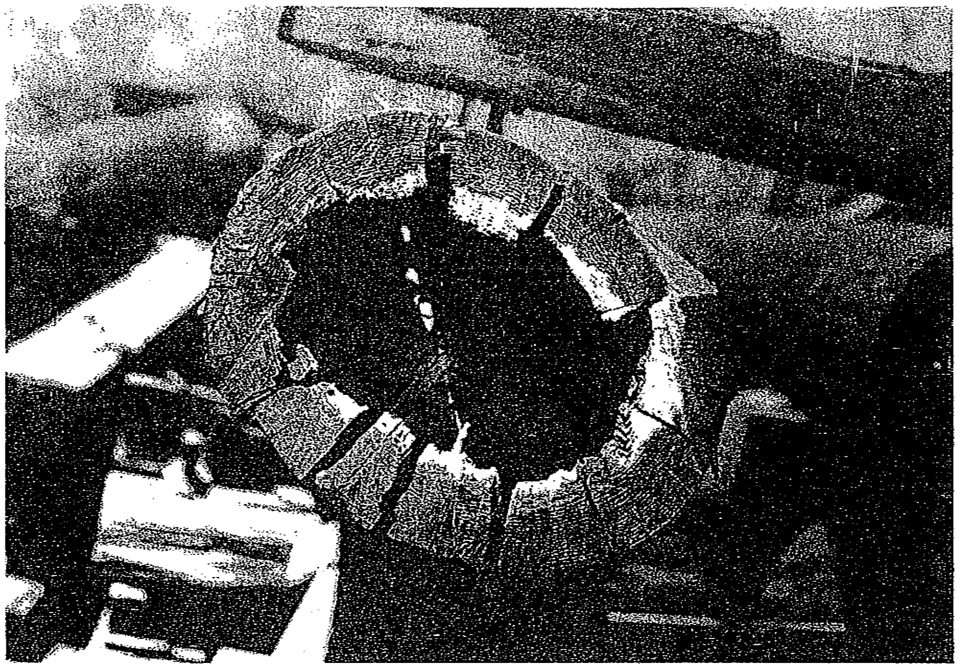


Figure 13-43.—Internal decay in a timber pile where it was drift-pinned to the cap. Before the breakage of the outside shell, caused by cap removal, the pile showed little exterior sign of the interior decay.

Portions of the substructure containing piers or bents use the same basic inspection criteria for the same potential problem areas as abutments. If these structures are in water, however, inspection is much more difficult because access is limited. In water locations, members are also more susceptible to mechanical damage from floating debris and ice. In shallow water, inspectors can wear hip-waders to examine exposed members, whereas in deeper water a small boat or float is required. When inspection below the water level is necessary, the service of a diver is required. Underwater inspections require a high degree of skill and must be well coordinated to accurately identify and record deficiencies.[7, 8]

For substructures located in seawater, low tides present the best opportunity to inspect for marine borer damage. Low-tide inspection is best suited for detecting *Limnoria,* which attack the external faces of members. A scraper and probe can be used to remove fouling organisms from the pile surface and thus permit better examination around bolt holes and adjoining wood members. Damage signs include an hourglass shape of piles in the tidal zone, bore holes; a general softening of wood in the attack areas; and loose bolts and bracing. Intertidal inspection is less effective for detecting damage by shipworms because they leave only a very small entrance hole on the wood surface, making visual detection difficult. Inspection methods using sonic instruments represent the best method for evaluating shipworm damage.

In areas where marine borer attack is suspected, an assessment of the hazard potential can be made by immersing sacrificial blocks of untreated wood at various depths around the substructure. These blocks are then removed periodically and examined for evidence of borer attack. Do not depend on the collection of driftwood to evaluate marine borer hazard because there is no way of knowing whether the wood came from sites outside the immediate area. Exposing wood samples can accurately assess marine borer hazard while providing a means for continually monitoring the long-term hazard.

Superstructure Inspection

After completing the substructure inspection, the inspector moves to the underside of the superstructure. It is best to thoroughly inspect all components from the bridge underside before moving to the roadway, since critical components are obscured by the wearing surface and deck. Superstructure inspection is generally hindered because access to the center portions of the underside is difficult or impossible without specialized equipment. When areas cannot be reached with ladders, a vehicle equipped with a mechanical arm or snooper may be required in order to adequately inspect the structure. Because ladders and other inspection equipment must be moved frequently to provide access to elevated areas, it is advisable that the inspection be performed by zones rather than by components. For the purposes of clarity, the following discussions are ordered by component.

Although most elements of the superstructure are out of ground contact, decay potential can be high in areas where water passes from the deck and collects at member interfaces, connections, checks, and crevices where air circulation and drying are inhibited. In many cases, this decay occurs with little or no surface evidence, although the member may be severely decayed inside. As a result, the inspector must be alert for conditions conducive to decay and must investigate areas where these conditions are likely to occur. As previously discussed, a moisture meter is a good tool for locating moisture conditions favorable to decay development (Figure 13-44). At least one boring should be made in areas of high moisture content where decay potential is considered highest. If decay is detected, additional borings should be taken to define its area, degree, and extent. If no decay is detected, but preservative penetration is shallow or moisture content is above 30 percent, it is desirable to remove a core for culturing to determine whether decay fungi are present.

The highest potential for decay in beams occurs at the deck-beam interface and attachment points, framing connections to other members, bearings, and seasoning checks. The deck-beam interface is one of the most frequent decay areas because water passing through the deck is trapped and enters fastener holes at the beam top. The hazard is highest when decks are attached with nails or lag screws that penetrate the top surface of the beam

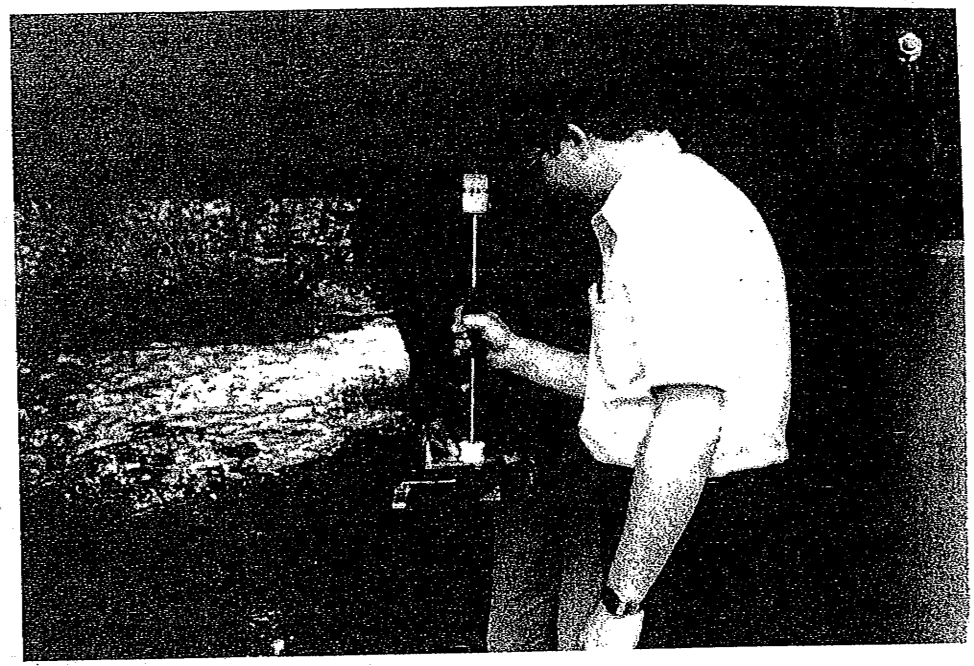


Figure 13-44.—The moisture content of a timber beam is measured with a resistance-type moisture meter.

(Figure 13-45). Glulam deck panels with bolted brackets do not involve attachments that penetrate the beam; thus, there is no significant increase in decay potential. On the deck underside, the inspector should be alert for signs of water movement and the presence of moisture at joint interfaces. Although stains are generally visible when water has passed through the deck, asphalt wearing surfaces tend to filter runoff, and visible signs are more difficult to detect. If significant decay is found along beam tops, it is advisable to remove deck sections to further examine beam condition.

In addition to the deck-beam interface and attachments, beam decay may develop in checks or delaminations, especially in the areas where end grain is exposed. Large checks or delaminations are not common in glulam and may be an indication of more severe structural problems. Bearings that trap water or show signs of beam crushing, and fasteners for transverse bracing or diaphragms are other potential decay locations. Sagging, splintering, or excessive deflections under load may also indicate mechanical damage or possible advanced decay. In some situations, surface decay may be present on a beam side or bottom that does not appear to be in an environment conducive to decay (Figure 13-46). Decay in such locations can occur in sawn lumber beams because of incomplete preservative penetration of heartwood.

Concurrent with beam inspection, the deck underside should be examined for signs of deterioration and conditions conducive to decay. Signs to observe include abnormal deflections and loose joints or fasteners, both of which may result from decay. Nail-laminated decks are frequently

Figure 13-45.—Severe decay in the tops of sawn lumber beams where the deck was attached to the beams with spikes.

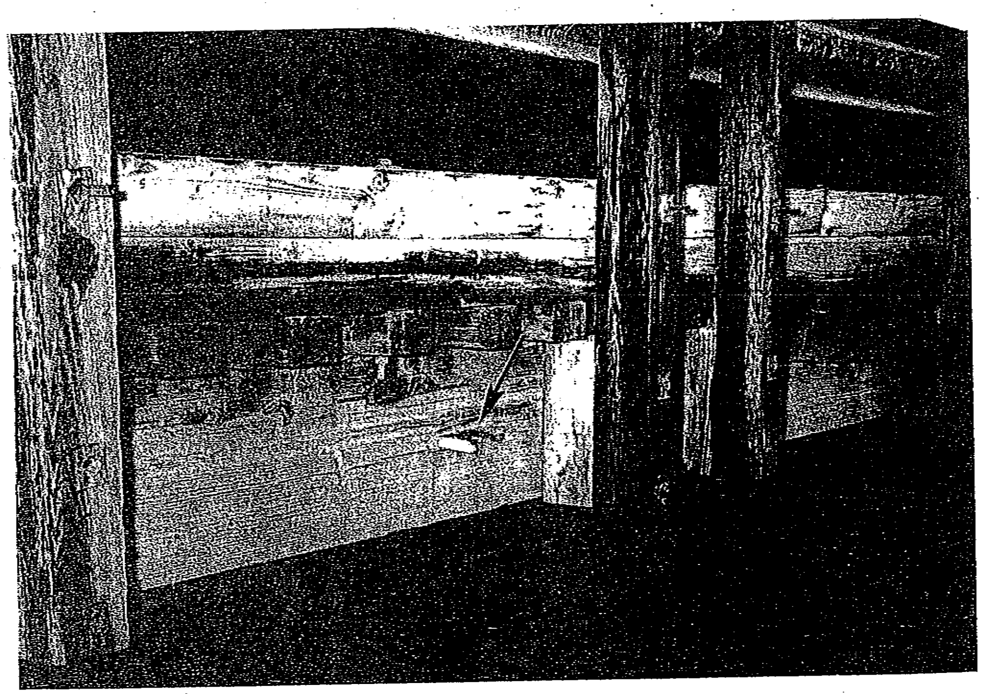


Figure 13-46.—Surface decay on the side of a sawn lumber beam (arrow). Decay in s[illegible] locations is usually the result of poor preservative penetration of the heartwood.

delaminated by dynamic loading. Although delamination may not adversely affect strength, it does create voids between laminations, allowing water to flow on supporting beams and other components. Susceptibility to internal deck decay is highest with nail-laminated lumber or plank decks because they are interconnected and/or attached with nails or spikes (Figure 13-47). All fabrication for glulam panels is generally done before preservative treatment and the decay potential is lower unless panels are attached with spikes, lag screws, or other fasteners placed after the deck is treated.

Figure 13-47.—Decay on the underside of a spike-attached lumber plank deck at the deck-beam interface (arrow).

When inspection of the bridge underside is complete, efforts are next directed to the roadway portion of the deck. The upper deck is subject to wear and abrasion from traffic, and the horizontal surface facilitates water and debris accumulation. The highest decay potential occurs at fasteners or zones of mechanical damage and is influenced by the degree of protection provided by the wearing surface. A partial wearing surface affords the least deck protection because the gap between the running surfaces traps debris and moisture. On watertight glulam or stress-laminated timber decks, standing water may accumulate between running planks and remain for long periods. Moisture is also trapped under steel plate or full-plank surfaces where penetrating fasteners are normally placed after deck treatment. Asphalt wearing surfaces do not use mechanical fasteners, but moisture can accumulate at the deck interface when the surface is cracked or otherwise broken from excessive deflection.

The moisture content of timber decks generally averages 20 percent, but may frequently be much higher.[35] The inspector should carefully check exposed deck surfaces for moisture content and other conditions conducive to decay. When deck moisture contents are high, it is advisable to remove a number of cores from sites near the fasteners and other high-hazard locations. If necessary, portions of the wearing surface should be removed to assess deck condition. If evidence of substantial deterioration is found, the entire wearing surface should be removed to thoroughly inspect the deck.

Timber rails and curbs (wheel guards) are some of the most exposed elements of the bridge superstructure, yet are often ignored in bridge inspection. Although they are not critical for support of the structure, they are important for user safety and should be thoroughly inspected. Rails and curbs are susceptible to weathering, seasoning checks, and vehicle impact or abrasion. Rails and curbs are commonly the last components installed during the construction process and their installation presents an increased potential for field cutting and boring to meet alignment requirements. The inspector should pay particular attention to fasteners and areas that trap water and debris. One very probable decay situation occurs when approach railposts are embedded in concrete (Figure 13-48).

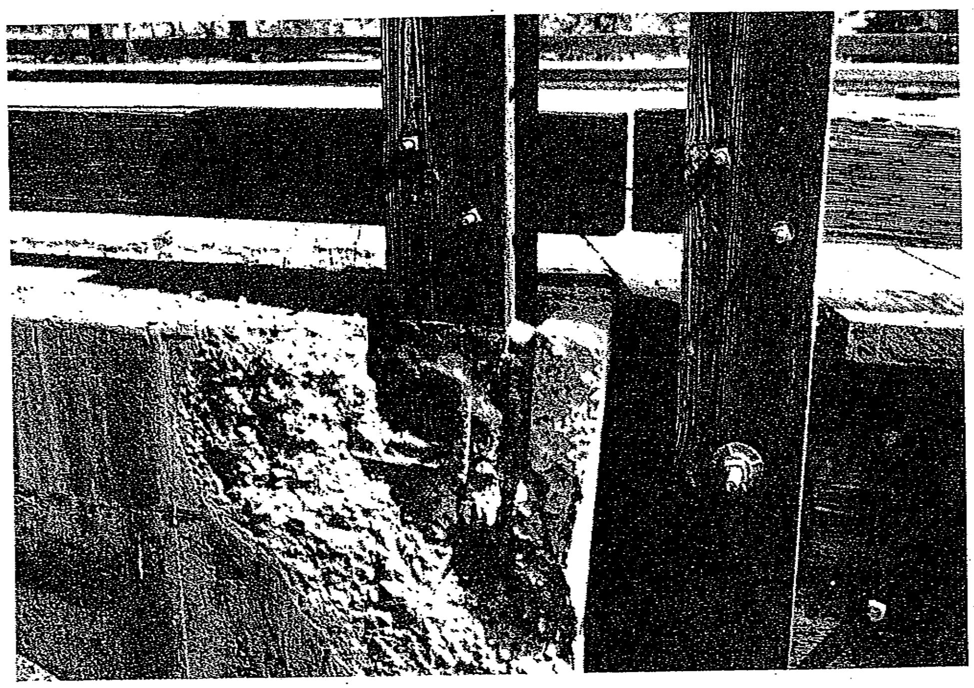


Figure 13-48.—Decay in a timber railpost embedded in concrete at the abutment. Concrete spalling was caused when water trapped in the post cavity was subjected to freeze-thaw cycles.

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SECTION 6—LOAD AND RESISTANCE FACTOR RATING

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LOAD AND RESISTANCE FACTOR RATING

6.1 INTRODUCTION

6.1.1 General

The load and resistance factor rating procedures of this section provide a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the *AASHTO LRFD Bridge Design Specifications*. The specific load ratings are used in identifying the need for load posting or bridge strengthening and in making overweight-vehicle permit decisions. Load ratings are routinely reported to the NBI for national bridge administration and are also used in local bridge management systems.

Bridge ratings are based on information in the bridge file, including the results of a recent field inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or loading noted during the inspection.

C6.1.1

There is a large inventory of existing bridges that have been rated using Allowable Stress or Load Factor methods. These ratings may be used for reporting existing bridges to the NBI while new bridges designed by LRFD should be reported using LRFR. Appendix D6.1 contains the other rating specifications.

6.1.2 Scope

This section provides procedures for the rating of bridges using the load and resistance factor philosophy. Procedures are presented for load rating bridges for the LRFD design loading, AASHTO and State legal loads, and overweight permit loads. These procedures are consistent in philosophy and approach of the *AASHTO LRFD Bridge Design Specifications*. The methodology is presented in a format using load and resistance factors that have been calibrated based upon structural reliability theory to achieve a minimum target reliability for the strength limit state. Guidance is provided on service limit states that are applicable to bridge load rating.

This Manual is intended for use in evaluating the types of highway bridges commonly in use in the United States that are subjected primarily to permanent loads and vehicular loads. Methods for the evaluation of existing bridges for extreme events such as earthquake, vessel collision, wind, flood, ice, or fire are not included herein. Rating of long-span bridges, movable bridges, and other complex bridges may involve additional considerations and loadings not specifically addressed in this Manual and the rating procedures should be augmented with additional evaluation criteria where required.

Specific provisions for the evaluation of curved girder steel bridges are not included in this Manual as criteria are still being developed for LRFD curved-girder design.

C6.1.2

The service limit states are not calibrated based upon reliability theory to achieve a target reliability, but are based on past practice. This Manual provides guidance to incorporate these traditional service limit states into the evaluation.

This Manual's primary focus is the assessment of the safety of bridges for live loads (including overloads) and fatigue. Extreme events have a very low probability of occurrence, but impart very high-magnitude forces on a structure. Study of past bridge failures indicates that failure due to hydraulics (scour/ice/debris) is the most common failure mode across the United States. Earthquake can also be a significant failure mode for bridges in regions considered to be seismically active. Bridges over navigable waterways with inadequate pier protection may be highly vulnerable to failure by vessel collision. The vulnerability to extreme events is an important bridge design consideration but it holds even greater significance in the overall safety assessment of existing bridges. It is important that Bridge Owners and evaluators recognize the vulnerabilities to these other failure modes so that a comprehensive safety assurance program may be developed for in-service bridges on a consistent and rational basis.

though the specification does not predict such distress, deviation from the governing specifications based upon the known behavior of the member under traffic may be used and shall be fully documented. Material sampling, instrumentation, and load tests may be helpful in establishing the load capacity for such members.

bridge types that use materials and details no longer in common use. However, the *AASHTO LRFD Bridge Design Specifications* incorporate the state-of-the-art in design and analysis methods, loadings, and strength of materials.

Specifications are calibrated documents in which the loads, load factors, and design methods are part of the whole and should not be separated. Combining factors contained in the original design specifications with those in the current LRFD design specifications should be avoided.

One of the purposes of this Manual is to provide guidance and data on older bridge types and materials that are not covered by the *AASHTO LRFD Bridge Design Specifications*, thereby allowing its application to a large inventory of existing bridges without having to resort to their original design specifications. The Manual seeks to extend the LRFD design philosophy for new bridges to the inventory of existing bridges in a consistent manner.

Evaluators are encouraged to research older materials and design methods as they provide valuable insight into the behavior of the country's older bridges.

6.1.6 Evaluation Methods

This Manual provides three methods for evaluating the safe maximum live-load capacity of bridges or for assessing their safety under a particular loading condition:

- Load and resistance factor rating of bridges,
- Load rating by load testing, and
- Safety evaluation using structural reliability methods for special cases.

Only the first method, load and resistance factor rating of bridges, is discussed in this section. Load testing and safety evaluation for special cases are discussed in Sections 8 and 9, respectively.

C6.1.6

Load testing may be used as an alternative method to directly assess the load capacity of a bridge when analytical methods of evaluation are not applicable or need verification.

Safety assessment of a bridge using structural reliability methods is intended for use in special cases where the uncertainty in load or resistance is significantly different from that assumed in this Manual.

6.1.7 Load and Resistance Factor Rating

Bridge evaluations are performed for varied purposes using different live-load models and evaluation criteria. Evaluation live-load models are comprised of the design live load, legal loads, and permit loads. This section specifies a systematic approach to bridge load rating for these load models, using the load and resistance factor philosophy, aimed at addressing the different uses of load rating results.

C6.1.7

Bridge load ratings are performed for specific purposes, such as: NBI and BMS reporting, local planning and programming, determining load posting or bridge strengthening needs, and overload permit review. Live load models, evaluation criteria, and evaluation procedures are selected based upon the intended use of the load rating results.

6.1.7.2 Legal Load Rating

This second level rating provides a single safe load capacity (for a given truck configuration) applicable to AASHTO and State legal loads. Live-load factors are selected based upon the truck traffic conditions at the site. Strength is the primary limit state for load rating; service limit states are selectively applied. The results of the load rating for legal loads could be used as a basis for decision making related to load posting or bridge strengthening.

6.1.7.3 Permit Load Rating

Permit load rating checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations. This is a third level rating that should be applied only to bridges having sufficient capacity for AASHTO legal loads. Calibrated load factors by permit type and traffic conditions at the site are specified for checking the load effects induced by the passage of the overweight truck. Guidance is also provided on the serviceability criteria that should be checked when reviewing permit applications.

6.1.8 Component-Specific Evaluation

6.1.8.1 Decks

Stringer-supported concrete deck slabs and metal decks that are carrying normal traffic satisfactorily need not be routinely evaluated for load capacity. The bridge decks should be inspected regularly to verify satisfactory performance. The inspection of metal decks should emphasize identifying the onset of fatigue cracks.

Timber decks that exhibit excessive deformations or deflections under normal traffic loads are considered suitable candidates for further evaluation and often control the rating. Capacity of timber plank decks is often controlled by horizontal shear.

C6.1.8.1

Test data indicates that the primary structural action of concrete decks is not flexure, but internal arching or membrane action. There is significant reserve strength in concrete decks designed by the AASHTO *Standard Specifications*. Heavily spalled and deteriorated concrete decks may be checked for punching shear under wheel loads.

6.1.8.2 Substructures

Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the Engineer has reason to believe that their capacity may govern the load capacity of the entire bridge.

Where deemed necessary by the Engineer, load rating of substructure elements and checking of

C6.1.8.2

Examples of distress that could trigger a load rating of substructure components include: a high degree of corrosion and section loss, changes in column end conditions due to deterioration, changes in column unbraced length due to scour, or columns with impact damage.

Special-emphasis inspection would entail a 100 percent hands-on visual inspection. Fracture-critical permanent loads and vehicular loads are considered to be of consequence in load rating. Environmental loads such as wind, ice, temperature, stream flow, and earthquake are usually not considered in rating except when unusual conditions warrant their inclusion.

6.2.2 Permanent Loads and Load Factors

The load rating of bridges shall consider all permanent loads. Permanent loads include dead loads and locked-in force effects from the construction process.

C6.2.2

Allowance for future wearing surface need not be provided in evaluation.

6.2.2.1 Dead Loads: *DC* and *DW*

The dead-load effects on the structure shall be computed in accordance with the conditions existing at the time of analysis. Dead loads should be based on dimensions shown on the plans and verified with field measurements. Where present, utilities, attachments, and thickness of wearing surface should be field verified at the time of inspection. Minimum unit weights of materials used in computing dead loads should be in accordance with LRFD Table 3-3, in the absence of more precise information.

C6.2.2.1

Care should be exercised in estimating the weight of concrete decks because significant variations of deck thickness have been found. Wearing surface thicknesses are also highly variable. Multiple measurements at curbs and roadway centerline should be used to determine an average wearing surface thickness.

6.2.2.2 Permanent Loads Other Than Dead Loads: *P*

Secondary effects from prestressing shall be considered as permanent loads.

C6.2.2.2

In continuous post-tensioned bridges, secondary moments are introduced as the member is stressed.

6.2.2.3 Load Factors

Load factors for permanent loads are as given in Table 6-1.

Where the permanent load increases the live load effects, the maximum load factor should be used. If the wearing surface thickness is field measured, γ_{DW} may be taken as 1.25.

A load factor of 1.0 shall be applied to the secondary effects from post-tensioning, cited in A6.2.2.2 ($\gamma_p = 1.0$).

6.2.3 Transient Loads

6.2.3.1 Vehicular Live Loads (Gravity Loads): *LL*

The nominal live loads to be used in the evaluation of bridges are selected based upon the purpose and intended use of the evaluation results. Live-load models for load rating include:

C6.2.3.1

The evaluation of bridge components to include the effects of longitudinal braking forces, specified in LRFD Article 3.6.4 in combination with dead- and live-load effects, should be done only where the

6.2.3.4 Pedestrian Live Loads: *PL*

Pedestrian loads on sidewalks need not be considered simultaneously with vehicular loads when load rating a bridge unless the Engineer has reason to expect that significant pedestrian loading will coincide with maximum vehicular loading. Pedestrian loads considered simultaneously with vehicular loads in calculations for load ratings shall be the probable maximum loads anticipated, but in no case should the loading exceed the value specified in LRFD Article 3.6.1.6.

6.2.3.5 Wind Loads: *WL* and *WS*

Wind loads need not be considered unless special circumstances justify otherwise.

C6.2.3.5

Wind loads are not normally considered in load rating. However, the effects of wind on special structures such as movable bridges, long-span bridges, and other high-level bridges should be considered in accordance with applicable standards.

6.2.3.6 Temperature Effects: *TG* and *TU*

Temperature effects need not be considered in calculating load ratings for non-segmental bridge components that have been provided with well-distributed steel reinforcement to control thermal cracking.

C6.2.3.6

Where temperature effects are considered, a reduced long-term modulus of elasticity for concrete may be used in the analysis.

Temperature gradient (*TG*) may be considered when evaluating segmental bridges.

6.2.3.7 Earthquake Effects: *EQ*

Earthquake effects need not be considered in calculating load ratings.

C6.2.3.7

In regions prone to seismic activity, the safety of bridges under earthquake loads may be evaluated in accordance with the provisions of *Seismic Retrofitting Manual for Highway Bridges;* FHWA-RD-94-052, May 1995.

6.2.3.8 Creep and Shrinkage: *CR* and *SH*

Creep and shrinkage effects do not need to be considered in calculating load ratings where there is well-distributed reinforcement to control cracking in non-segmental, non-prestressed components.

6.3 STRUCTURAL ANALYSIS

6.3.1 General

Methods of structural analysis suitable for the evaluation of bridges shall be as described in Section 4 of the *AASHTO LRFD Bridge Design Specifications* and in this section.

C6.3.1

Evaluation seeks to verify adequate performance of existing bridges with an appropriate level of effort. Within a given evaluation procedure, the evaluator has the option of using simplified methods that tend to be somewhat conservative or pursue a more refined approach for improved accuracy. It is recommended that wherever feasible, The distribution factor is generally lower for increased gage widths.

Exterior girders of existing bridges may have been designed for less capacity than the interior girders. Additionally, they may also be subjected to increased deterioration due to their increased environmental exposure. Approximate methods of analysis for exterior girders are often less reliable than interior girders due to the structural participation of curbs and parapets. The level of structural participation could vary from bridge to bridge. Field testing (load testing) procedures described in Section 8 may be employed to verify the behavior of exterior girders.

Prestressed concrete adjacent box-beam and slab bridges built prior to 1970 may not have sufficient transverse post-tensioning (LRFD C4.6.2.2.1 requires a minimum prestress of 0.25 ksi) to act as a unit. These bridges should be analyzed using the S/D method of live-load distribution provided in the *AASHTO LRFD Bridge Design Specifications*.

6.3.3 Refined Methods of Analysis

Bridges that exhibit insufficient load capacity when analyzed by approximate methods, and bridges or loading conditions for which accurate live-load distribution formulas are not readily available may be analyzed by refined methods of analysis as described in LRFD Article 4.6.3.

When a refined method of analysis is used, a table of distribution factors for extreme force effects in each span should be provided in the load rating report to aid in future load ratings.

C6.3.3

Some cases where refined analysis methods would be considered appropriate include:

- Girder spacings and span lengths outside the range of LRFD-distribution formulas.
- Varying skews at supports.
- Curved bridges.
- Low-rated bridges.
- Permit loads with nonstandard gage widths and large variations in axle configurations.

Many older bridges have parapets, railings, and curbs that are interrupted by open joints. The stiffness contribution of these elements to bridge response should be verified by load testing, if they are to be included in a refined analysis.

Most analytical models are based on linear response, where load effect is proportional to the load applied. Conversely, the resistance models used for design and evaluation assume nonlinear response at the strength limit state. The rationale for this inconsistency is found in the "lower bound theorem" which states that for a structure that behaves in a ductile manner the collapse load computed on the basis of an assumed equilibrium moment diagram is less than or equal to the true ultimate collapse load. Restated in simpler terms, the theorem implies that as long as the requirements of ductility and equilibrium are satisfied, the exact

6.4.2 General Load-Rating Equation

6.4.2.1 General

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_p)(P)}{(\gamma_L)(LL + IM)}$$

Eq. (6-1)

For the Strength Limit States:

$$C = \varphi_c\, \varphi_s\, \varphi\, R_n$$

Where the following lower limit shall apply:

$$\varphi_c\, \varphi_s \geq 0.85$$

For the Service Limit States:

$$C = f_R$$

Where:

RF	=	Rating factor
C	=	Capacity
f_R	=	Allowable stress specified in the LRFD code
R_n	=	Nominal member resistance (as-inspected)
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live-load effect
IM	=	Dynamic load allowance
γ_{DC}	=	LRFD load factor for structural components and attachments
γ_{DW}	=	LRFD load factor for wearing surfaces and utilities
γ_p	=	LRFD load factor for permanent loads other than dead loads = 1.0
γ_L	=	Evaluation live-load factor
φ_c	=	Condition factor
φ_s	=	System factor
φ	=	LRFD resistance factor

The load rating shall be carried out at each applicable limit state and load effect with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 6-1.

C6.4.2.1

It should be noted that load modifiers (η) relating to ductility, redundancy, and operational importance contained in the *AASHTO LRFD Bridge Design Specifications* (Article 1.3.2.1) are not included in the general load-rating equation. In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factor φ_s. Operational importance is not included as a factor in the load rating provisions of this Manual.

The load rating of a deteriorated bridge should be based on a recent thorough field inspection. Only sound material should be considered in determining the nominal resistance of the deteriorated section. Load ratings may also be calculated using as-built member properties to serve as a baseline for comparative purposes.

Resistance factor φ has the same value for new design and for load rating. Also, $\varphi = 1.0$ for all nonstrength limit states. For condition factors, see Article 6.4.2.3. For system factors, see Article 6.4.2.4.

6.4.2.3 Condition Factor: φ_c

Use of Condition Factors as presented below may be considered optional based on an agency's load-rating practice.

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 6-2 Condition Factor: φ_c.

Structural Condition of Member	φ_c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

C6.4.2.3

The uncertainties associated with the resistance of an existing intact member are at least equal to that of a new member in the design stage. Once the member experiences deterioration and begins to degrade, the uncertainties and resistance variabilities are greatly increased (scatter is larger).

Additionally, it has been observed that deteriorated members are generally prone to an increased rate of future deterioration when compared to intact members. Part of φ_c relates to possible further section losses prior to the next inspection and evaluation.

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in identifying the true extent of deterioration for use in calculating the nominal member resistance. If section properties are obtained accurately, by actual field measurement of losses rather than by an estimated percentage of losses, the values specified for φ_c in Table 6-2 may be increased by 0.05 ($\varphi_c \leq 1.0$).

The condition factor, φ_c, tied to the structural condition of the member, accounts for the member deterioration due to natural causes (i.e., atmospheric corrosion). Damage caused by accidents is specifically not considered here.

If condition information is collected and recorded in the form of NBI condition ratings only (not as element level data), then the following approximate conversion may be applied in selecting φ_c.

Table C6-1 Approximate Conversion in Selecting φ_c.

Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition
6 or higher	Good or Satisfactory
5	Fair
4 or lower	Poor

6.4.2.4 System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6-3 are more conservative than the LRFD-design values and may be used at the

C6.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as non-redundant.

If Table 6-3 is used, the system factors are used to maintain an adequate level of system safety. Non-

6.4.3 Design-Load Rating

6.4.3.1 Purpose

The design-load rating assesses the performance of existing bridges utilizing the LRFD-design loading (HL-93) and design standards. The design-load rating of bridges may be performed at the same design level (Inventory level) reliability adopted for new bridges by the *AASHTO LRFD Bridge Design Specifications* or at a second lower-level reliability comparable to the Operating level reliability inherent in past load-rating practice. The design-load rating produces Inventory and Operating level rating factors for the HL-93 loading.

The design-load rating serves as a screening process to identify bridges that should be load rated for legal loads, per the following criteria:

- Bridges that pass HL-93 screening at the Inventory level will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the *AASHTO LRFD Bridge Design Specifications.*

- Bridges that pass HL-93 screening only at the Operating level will have adequate capacity for AASHTO legal loads, but may not rate (RF > 1) for all State legal loads, specifically those vehicles significantly heavier than the AASHTO trucks.

The results are also suitable for use in NBI reporting, and bridge management and bridge administration, at a local or national level. The rating results for service and fatigue limit states could also guide future inspections by identifying vulnerable limit states for each bridge.

C6.4.3.1

The design-load rating is performed using dimensions and properties for the bridge in its present condition, obtained from a recent field inspection.

No further evaluation is necessary for bridges that have adequate capacity ($RF > 1$) at the Inventory level reliability for HL-93. Bridges that pass HL-93 screening only at the Operating level reliability will not have adequate capacity for State legal loads significantly heavier than the AASHTO legal loads. Existing bridges that do not pass a design-load rating at the Operating level reliability should be evaluated by load rating for AASHTO legal loads using procedures provided in this section.

6.4.3.2 Live Loads and Load Factors

6.4.3.2.1 Live Load

The LRFD-design, live-load HL-93 (see Appendix B.6.1) shall be used.

6.4.3.2.2 Live-Load Factors

The evaluation live-load factors for the Strength I limit state shall be taken as shown in Table 6-4.

Table 6-4 Load Factors for Design Load: γ_L.

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

C6.4.3.2.2

Service limit states that are relevant to design-load rating are discussed under the articles on resistance of structures (see Sections 6.5, 6.6, and 6.7).

- For negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 multiplied by 0.75 heading in the same direction separated by 30 ft.

In addition, for span lengths greater than 200 ft., critical load effects shall be created by:

- AASHTO Type 3-3 multiplied by 0.75 and combined with a lane load of 0.2 klf.

Dynamic load allowance shall be applied to the AASHTO legal vehicles and not the lane loads. If the ADTT is less than 500, the lane load may be excluded and the 0.75 factor changed to 1.0 if, in the Engineer's judgment, it is warranted.

AASHTO legal vehicles, designated as Type 3, Type 3S2, and Type 3-3 are sufficiently representative of average truck configurations in use today, and are used as vehicle models for load rating. These vehicles are also suitable for bridge posting purposes. Load ratings may also be performed for State legal loads that have only minor variations from the AASHTO legal loads using the live-load factors provided in Table 6-5 for the AASHTO vehicles. It is unnecessary to place more than one vehicle in a lane for spans up to 200 ft. because the load factors provided have been modeled for this possibility.

The federal bridge formula (Reference: TRB Special Report 225, *Truck Weight Limits Issues and Options*, 1990) restricts truck weights on interstate highways through (a) a total, or gross, vehicle weight limit of 80 kips; (b) limits on axle loads (20 kips for single axles, 34 kips for tandem axles); and (c) a bridge formula that specifies the maximum allowable weight on any group of consecutive axles. based on the number of axles in the group and the distance from first to the last axles. Grandfather provisions in the federal statutes allow states to retain higher limits than these if such limits were in effect when the applicable federal statutes were first enacted.

The objective of producing new *AASHTO LRFD Bridge Design Specifications* that will yield designs having uniform reliability required as its basis a new live-load model with a consistent bias when compared with the exclusion vehicles. The model consisting of either the HS-20 truck plus the uniform lane load or the tandem plus the uniform lane load (designated as HL-93 loading) resulted in a tight clustering of data around a 1.0 bias factor for all force effects over all span lengths. This combination load was therefore, found to be an adequate basis for a notional design load in the *AASHTO LRFD Bridge Design Specifications.*

While this notional design load provides a convenient and uniform basis for design and screening of existing bridges against new bridge safety standards, it has certain limitations when applied to evaluation. The notional design load bears no resemblance or correlation to legal truck limits on the roads and poses practical difficulties when applied to load rating and load posting of existing bridges.

A characteristic of the AASHTO family of legal loads (Type 3, Type 3S2, Type 3-3) is that the group satisfies the federal bridge formula. The AASHTO legal loads model three portions of the bridge formula which control short, medium, and long spans. Therefore, the combined use of these three AASHTO legal loads results in uniform bias over all span lengths, as achieved with the HL-93 notional load model (see Appendix B6.4). These vehicles are presently widely used for load rating

The live-load factors in Table 6-5 were determined, in part, by reducing the target beta level from the design level of 3.5 to the corresponding operating level of 2.5, according to NCHRP Report 454. Several parametric analyses indicate this reduction in beta corresponds to a reduced load factor ratio of about 0.76 (i.e., 1.35/1.75). Thus, the load factors in Table 6-5 have been calibrated to represent an equivalent Operating level of loading. Therefore, it is reasonable to increase the load factor up to the design target beta level (or equivalent Inventory level of loading), if the Engineer deems appropriate, by multiplying by the reciprocal of 0.76 or 1.3.

Site-Specific, Live-Load Factors

Consideration should be given to using site-specific load factors when a bridge on a low-volume road may carry unusually heavy trucks or industrial loads due to the proximity of the bridge to an industrial site.

When both truck weight and truck traffic volume data are available for a specific bridge site, appropriate load factors can be derived from this information. Truck weights at a site should be obtained by generally accepted weigh-in-motion technology. In general, such data should be obtained by systems able to weigh all trucks without allowing heavy overweight vehicles to bypass the weighing operation.

To obtain an accurate projection of the upper tail of the weight histogram, only the largest 20 percent of all truck weights are considered in a sample for extrapolating to the largest loading event. A sufficient number of truck samples need to be taken to provide accurate parameters for the weight histogram.

For a two- or more than two-lane loading case, the live-load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{2W^* + t_{(\mathrm{ADTT})}\,1.41\sigma^*}{240}\right] > 1.30 \qquad \text{Eq. (C6-1)}$$

For the single-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{W^* + t_{(\mathrm{ADTT})}\,\sigma^*}{120}\right] > 1.80 \qquad \text{Eq. (C6-2)}$$

where:

W^* = Mean truck weight for the top 20 percent of the weight sample of trucks (kips) fluctuations. The data collection period should be sufficient to capture at least 400 trucks in the upper 20 percent of the weight sample for the site. Additional guidance on determining site-specific load factors can be found in the NCHRP Report 454.

6.4.4.3 Dynamic Load Allowance: *IM*

The static effects of the truck loads shall be increased by 33 percent for strength and service limit states to account for the dynamic effects due to moving vehicles. The dynamic load allowance shall be applied only to the axle loads when the lane type load given in Appendix B.6.2 is used for evaluation.

The dynamic load allowance for the evaluation of wood components shall be reduced to 50 percent of the values specified.

C6.4.4.3

The factor to be applied to the static load effects shall be taken as: $(1+IM/100)$. The factors are applicable to simple and continuous span configurations.

The dynamic response of a bridge to a crossing vehicle is a complex problem affected by the pavement surface conditions and by the dynamic characteristics of both the bridge and vehicle. In the majority of bridge load tests, roadway imperfections, and irregularities were found to be a major factor influencing bridge response to traffic loads. The 33 percent dynamic load allowance specified deliberately reflects conservative conditions that may prevail under certain distressed approach and bridge deck conditions with bumps, sags, or other major surface deviations and discontinuities. In longitudinal members having spans greater than 40 ft. with less severe approach and deck surface conditions, the dynamic load allowance (*IM*) may be decreased as given below in Table C6-3:

Table C6-3 Dynamic Load Allowance: *IM*.

Riding Surface Conditions	*IM*
Smooth riding surface at approaches, bridge deck, and expansion joints	10%
Minor surface deviations or depressions	20%

Providing a dynamic load allowance primarily as a function of pavement surface conditions is considered a preferred approach for evaluation. Pavement conditions that were not known to the designer are apparent to the inspector/evaluator. The riding surface conditions used in Table C6-3 are not tied to any measured surface profiles, but are to be selected based on field observations and judgment of the evaluator. Condition of deck joints and concrete at the edges of deck joints affect rideability and dynamic forces induced by traffic. Inspection should carefully note these and other surface discontinuities in order to benefit from a reduced dynamic load allowance.

The dynamic load allowance for components determined by field testing may be used in lieu of values specified herein. The use of full-scale dynamic testing under controlled or normal traffic conditions remains the most reliable way of

6.4.5.2 Purpose

Section 6.4.5 provides procedures for checking bridges to determine the load effects induced by the overweight permit loads and their capacity to safely carry these overloads. Permit load rating should be used only if the bridge has a rating factor greater than 1.0 when evaluated for AASHTO legal loads.

C6.4.5.2

Permit vehicles should be rated by using load-rating procedures given in Section 6.4.5, with load factors selected based upon the permit type, loading condition, and site traffic data. The live load to be used in the load-rating equation for permit decisions shall be the actual permit vehicle weight and axle configuration.

The factors recommended for evaluating permit loads are calibrated with the assumptions that the bridge, as a minimum, can safely carry AASHTO legal loads, as indicated by the evaluation procedures given in Article 6.4.4. This requirement is especially evident when using reduced live-load factors for permits based on a small likelihood that there will be multiple presence of more than one heavy vehicle on the span at one time. Such multiple presence situations are considered in the calibration of the checking equations of both the *AASHTO LRFD Bridge Design Specifications* and the evaluation procedures given in this Manual.

6.4.5.3 Permit Types

6.4.5.3.1 Routine (Annual) Permits

Routine permits are usually valid for unlimited trips over a period of time, not to exceed one year. The permit vehicles may mix in the traffic stream and move at normal speeds without any movement restrictions.

6.4.5.3.2 Special (Limited Crossing) Permits

Special permits are usually valid for a single trip only or for a limited number of trips. These permit vehicles are usually heavier than those vehicles issued routine permits.

Single-trip permits are good for only one trip during a specified period of time (typically 3–5 days). Multiple-trip permits grant permission to transport overweight shipments during a 30–90 day period.

Single-trip permits for excessively heavy loads may have certain conditions and restrictions imposed to reduce the load effect, including, but not limited to:

- Requiring the use of escorts to restrict all other traffic from the bridge being crossed.

- Requiring the permit vehicle to be in a certain position on the bridge (e.g., in the center or to one side) to reduce the loading on critical components.

C6.4.5.3.2

Upper limit of 100 special permit crossings was used for calibration purposes in this Manual. Permits operating at a higher frequency should be evaluated as routine permits.

Table 6-6 Permit load Factors: γ_L.

Permit Type	Frequency	Loading Condition	DF^a	ADTT (one direction)	Load Factor by Permit Weight[b] Up to 100 kips	Load Factor by Permit Weight[b] ≥ 150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge next to it	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

Notes:

[a] DF = LRFD-distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and ADTT value. Use only axle weights on the bridge.

The live-load distribution analysis for routine permits is done using LRFD two-lane distribution factors which assume the simultaneous side-by-side presence of two equally heavy vehicles in each lane. This condition is too conservative for permit load analysis. The live-load factors herein were derived to account for the possibility of simultaneous presence of non-permit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher ADTTs and lower for heavier permits. The live-load factors in Table 6-6 for routine permits must be applied together with the upper limit of permit weights operating under a single permit and the corresponding two-lane distribution factor.

For situations where the routine permit is below 100 kips, the live-load factors are the same as those given for evaluating legal loads. This requirement reflects the fact that in a traffic stream, the presence of random, heavy, overloaded vehicles may control the extreme loading case when compared to permit weights, which are close to the limit of 80 kips. When the routine permit weight is above 100 kips, then the live-load factors are reduced as shown in Table 6-6. This reduction reflects the lower probability of two simultaneously heavy vehicles equal to the permit weight crossing the span at the same instant (LRFD two-lane distribution factor assumes that an identical vehicle is simultaneously present in each lane). The calibration of these live-load factors for routine permits uses the same traffic statistics used in calibrating the *AASHTO LRFD Bridge Design Specifications* as well as the evaluation factors

6.4.5.5 Dynamic Load Allowance: *IM*

The dynamic load allowance to be applied for permit load rating shall be as specified in Article 6.4.4.3 for legal loads, except that for slow moving (≤10 mph) permit vehicles the dynamic load allowance may be eliminated.

6.4.5.6 Exterior Beams

Permit load factors given in Table 6-6 are applicable to both interior and exterior beam ratings. Distribution of live load to exterior beams as defined in LRFD Article 4.6.2.2.2d shall apply with the following modifications:

- For special permits, use a one-lane loaded condition only. Where a one-lane loaded condition is assumed, the LRFD multiple presence factor need not be applied (the built-in multiple presence factor in the LRFD one-lane distribution factor should be divided out).
- For routine permits, a multi-lane loaded condition shall be assumed. Permit trucks of equal weights shall be assumed to be present in each lane in determining the governing distribution factor.

C6.4.5.6

In LRFD, live-load distribution to the exterior beams for bridges with diaphragms or cross-frames must be checked by an additional investigation that assumes rigid body behavior of the section, per LRFD Article 4.6.2.2.2d.

6.4.5.7 Continuous Spans

Closely spaced heavy axles can cause uplift in end spans of continuous bridges. During permit reviews, uplift in continuous span bridges and its effect upon bearings should be considered.

C6.4.5.7

When the upward *LL* reaction reduces the total reaction to less than 10 percent of normal downward *DL* reaction, uplift may be considered to occur. Unless the uplift is counteracted (by weights or tie-downs), the vehicle should not be permitted on the bridge.

6.5 CONCRETE STRUCTURES

6.5.1 Scope

The provisions of Section 6.5 apply to the evaluation of concrete bridge components reinforced with steel bars and/or prestressing strands or bars. The provisions of this section combine and unify the requirements for reinforced and prestressed concrete.

6.5.2 Materials

6.5.2.1 Concrete

When the compressive strength of concrete, f'_c, is unknown and the concrete is in satisfactory condition, f'_c, for reinforced concrete superstructure

C6.5.2.1

Cores may also be taken where the initial load capacity based on design concrete strength is considered inadequate. Concrete strength may have

6.5.4 Limit States

The applicable limit states and their load combinations for the evaluation of concrete members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6-1 and in these articles.

Table 6-9 Tensile Strength of Prestressing Strand.

Year of Construction	Tensile Strength, f_{pu} ksi
Prior to 1963	232.0
1963 and later	250.0

6.5.4.1 Design-Load Rating

The Strength I load combinations shall be checked for reinforced concrete components. The Strength I and Service III load combinations shall be checked for prestressed concrete components.

C6.5.4.1

Service III need not be checked for HL-93 at the Operating level as Service III is a Design-level check for crack control in prestressed components.

The Service I load combination of the *AASHTO LRFD Bridge Design Specifications* need not be checked for reinforced concrete bridges, as it pertains to the distribution of reinforcement to control crack widths in reinforced concrete beams. Distribution of reinforcement for crack control is a design criterion that is not relevant to evaluation. In LRFD, Service I is also used to check compression in prestressed concrete bridges. This check may govern at prestress transfer, but usually will not govern live-load capacity under service conditions.

Most prestressed designs are designed for no cracking under full-service loads. Fatigue is not a concern until cracking is initiated. Hence, prestressed components need not be routinely checked for fatigue.

Rating factors for applicable limit states computed during design-load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

6.5.4.2 Legal load Rating and Permit load Rating

Load ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength- and service limit states, guided by considerations presented in these articles.

6.5.4.2.1 Strength Limit State

Concrete bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

that materials and construction are of good quality and there is no loss of material design strength, or, when warranted, the material strength has been established by testing, and any reductions in area due to deterioration have been considered.

matrix and the separation of aggregates due to chemical agents or other causes. In such cases, material sampling and testing should be considered to assess concrete strength and quality.

Deterioration of concrete components does not necessarily reduce their flexural resistance. The actual amount of capacity reduction depends on the type of deterioration and its location. Loss in reinforcing steel area at a critical location could directly affect the moment capacity as concrete beams are normally designed as under-reinforced sections. However, loss of cover due to spalling does not have a significant influence on the flexural resistance, if the ends of the reinforcing steel are properly anchored. Spalls on the compression side could change the internal moment-arm leading to a slight reduction in load capacity.

6.5.6 Maximum Reinforcement

The area of prestressed and non-prestressed reinforcement to be used in calculating flexural resistance shall not exceed the maximum amount permitted in LRFD Article 5.7.3.3.1.

C6.5.6

Load rating of over-reinforced sections should utilize only the area of reinforcement not exceeding the limiting value for maximum reinforcement.

6.5.7 Minimum Reinforcement

Concrete members that do not satisfy the minimum flexural reinforcement provisions of LRFD Article 5.7.3.3.2 shall have their flexural resistance reduced by multiplying by a reduction factor K, where:

$$K = \frac{M_r}{M_{\min}} \qquad \text{Eq. (6-3)}$$

where: $M_{\min}$ = Lesser of $1.2\ M_{cr}$ or $1.33\ M_u$

M_{cr} = Cracking Moment

$$M_{cr} = (f_r + f_{pb})\,S_{bc} - M_{d,nc}\left(\frac{S_{bc}}{S_b} - 1\right) \qquad \text{Eq. (6-4)}$$

where:

f_r = Modulus of rupture (LRFD Article 5.4.2.6)

f_{pb} = Compressive stress in concrete due to effective prestress in the precompressed tensile zone. Effective prestress for standard prestressed members may be computed using approximate lump sum estimate of time-dependent losses as provided in LRFD Article 5.9.5.3

$M_{d,nc}$ = Non-composite dead-load moment

C6.5.7

The equation for M_{cr} is adapted from the AASHTO *Standard Specifications,* Article 9.18.2.1. Where beams are designed to be non-composite, substitute S_b for S_{bc}.

quite low. Typically, locations near the 0.25 point could be critical because of relatively high levels of both shear and moment. Also contributing to the need for checking multiple locations along the beam is the fact that the stirrup spacings are typically not constant, but vary.

6.5.10 Secondary Effects from Prestressing

Secondary effects from prestressing shall be considered as permanent loads with load factors as cited in Article 6.2.2.2.

C6.5.10

Reactions are produced at the supports in continuous spans under post-tensioning loads, giving rise to secondary moments in the girders. The secondary moments are combined with the primary moments to provide the total moment effect of the post-tensioning.

6.5.11 Concrete Bridges with Unknown Reinforcement

A concrete bridge for which the details of the reinforcement are not known, need not be posted for restricted loading if it has been carrying normal traffic for an appreciable period and shows no distress. The bridge shall be inspected regularly (see Appendix A.6.1.4) to verify satisfactory performance.

C6.5.11

Bridge Owners may consider nondestructive proof load tests to establish a safe load capacity for such bridges. Knowledge of the live loading used in the original design and the current condition of the structure may also provide a basis for assigning a safe load capacity.

6.5.12 Temperature, Creep, and Shrinkage Effects

Temperature, creep, and shrinkage effects need not be considered at the strength limit state for components that have been provided with well-distributed steel reinforcement to control cracking.

C6.5.12

Temperature, creep, and shrinkage are primarily strain inducing effects. As long as the section is ductile, such changes in strain are not expected to cause failure. Where temperature cracks are evident and analysis is considered warranted, temperature effects due to time-dependent fluctuations in effective bridge temperature may be treated as long-term loads, with a long-term modulus of elasticity of concrete reduced to one-third its normal value.

6.5.13 Prestressed Concrete Beams Made Continuous in the Field

The capacity of the mild steel reinforcement over the piers should be checked for the strength limit state as it could control load ratings for this type of structure.

6.6 STEEL STRUCTURES

6.6.1 Scope

The provisions of Section 6.6 shall apply to the evaluation of steel and wrought-iron components of bridges.

6.6.2.2 Pins

If the material designation for pins is unknown, the yield strength may be selected from Table 6-12, based on the year of construction.

Table 6-12 Minimum Yield Point of Pins by Year of Construction.

Year of Construction	Minimum Yield Point, F_y ksi
Prior to 1905	25.5
1905 through 1935	30
1936 through 1963	33
After 1963	36

6.6.2.3 Wrought Iron

When the material designation is unknown for wrought iron, the minimum tensile strength, F_u, should be taken as 48 ksi and the minimum yield point, F_y, should be taken as 26 ksi.

Where practical, coupon tests should be performed to confirm the minimum mechanical properties used in the evaluation.

6.6.3 Resistance Factors

Resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Article 6.5.4.2.

C6.6.3

For service limit states, $\varphi = 1.0$.

6.6.4 Limit States

The applicable limit states and their load combinations for the evaluation of structural steel and wrought iron members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6-1 and in these articles.

6.6.4.1 Design-Load Rating

Strength I and Service II load combinations shall be checked for the design loading. Live-load factors shall be taken as tabulated in Table 6-1.

In situations where fatigue-prone details are present (category C or lower) a rating factor for infinite fatigue life should be computed. Members that do not satisfy the infinite fatigue life check may be evaluated for remaining fatigue life using procedures given in Section 7 of this Manual. This is an optional requirement.

C6.6.4.1

Rating factors for applicable strength, service, and fatigue limit states computed during the design load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

fitting to load-factor operating level serviceability rating, the built-in conservatism in the distribution analysis is considered appropriate. Escorted special permits operating with no other vehicles on the bridge may be analyzed using one-lane distribution factors for Service II.

The stress limitation of $0.8F_{yf}$ for the negative moment region of composite spans with longitudinal reinforcement has been found to be conservative. The Autostress design method places no restriction on the maximum stress due to negative moment at overload. Continuous span bridges are allowed to shake down and respond to subsequent overloads in an elastic manner. This can also be applied to the rating of existing bridges. It is reasonable to assume that any bridge that has been in service for some years would have undergone the shakedown process.

6.6.5 Effects of Deterioration on Load Rating

A deteriorated structure may behave differently than the structure as originally designed and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

C6.6.5

Tension Members with Section Losses Due to Corrosion

Corrosion loss of metals can be uniform and evenly distributed or it can be localized. Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Since localized corrosion results in irregular localized reductions in area, a simplified approach to evaluating the effects of localized corrosion is to consider the yielding of the reduced net area as the governing limit state. Due to their self-stabilizing nature, stress concentrations and eccentricities induced by asymmetrical deterioration may be neglected when estimating the tension strength of members with moderate deterioration.

For eyebars and pin plates, the critical section is located at the pin hole normal to the applied stress. In evaluating eyebars with significant section loss in the head, the yielding of the reduced net section in the head should be checked as it may be a governing limit state.

Deterioration of lacing bars and batten plates in built-up tension members may affect the load sharing among the main tension elements at service loads. At ultimate load yielding will result in load redistribution among the tension elements and the effect on capacity is less significant.

Compression Members with Section Losses Due to Corrosion

a) Uniform Corrosion

Local Effects—The susceptibility of members with reduced plate thickness to local buckling should be evaluated with respect to the limiting width/thickness ratios specified in LRFD batten plates has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

Flexural Members with Section Losses Due to Corrosion

Uniform Corrosion

The reduction in bending resistance of laterally supported beams with stiff webs will be proportional to the reduction in section modulus of the corroded cross-section compared to the original cross-section. Either the elastic or plastic section modulus shall be used, as appropriate. Local and overall beam stability may be affected by corrosion losses in the compression flange.

The reduction in web thickness will reduce shear resistance and bearing capacity due to both section loss and web buckling. When evaluating the effects of web losses, failure modes due to buckling and out-of-plane movement that did not control their original design may govern. The loss in shear resistance and bearing capacity is linear up to the point where buckling occurs.

Localized Corrosion

Small web holes due to localized losses not near a bearing or concentrated load may be neglected. All other web holes should be analytically investigated to assess their effect.

A conservative approach to the evaluation of tension and compression flanges with highly localized losses is to assume the flange is an independent member loaded in tension or compression. When the beam is evaluated with respect to its plastic moment capacity, the plastic section modulus for the deteriorated beam may be used for both localized and uniform losses.

6.6.6 Tension Members

Members and splices subjected to axial tension shall be investigated for yielding on the gross section and fracture on the net section as specified in LRFD Article 6.8.2.

6.6.6.1 Links and Hangers

The following provisions are given for the evaluation of pin-connected tension members other than eyebars:

1) The net section through the pin hole transverse to the axis of the member shall be 40 percent greater than the net section of the main member.

C6.6.6.1

Design of pin and hanger connections assumes free rotation at the pin. Accumulation of dirt and corrosion developed between the elements of the pin and hanger assembly could result in unintended partial or complete fixity of the pin and hanger connection. Very large in-plane bending stresses in the hangers and torsional stresses in the pins could be expected from rotational fixity. The fatigue life of the hangers could also be reduced. Build-up of sides using either stay plates in combination with single or double lacing, perforated cover plates, or batten plates. To allow for the reduced strength of batten plate compression members only, the actual length of the member shall be multiplied by the adjustment factor given in Table 6-13 to obtain the adjusted value of L/r to be used in computing the column slenderness factor λ.

For compression members having a solid plate on one side and batten plates on the other, the forgoing factors shall be reduced 50 percent.

Adjusted L/r (batten plate both sides) β

$$= \text{Actual } L/r \times \text{factor}$$

Adjusted L/r (batten plate one side)

$$= \text{Actual } L/r \times [1 + 1/2\ (\text{factor} - 1)]$$

6.6.8 Combined Axial Compression and Flexure

The load rating of steel members subjected to axial compression and concurrent moments, such as arches and beam-columns, shall be determined using the interaction equations specified in LRFD Article 6.9.2.2.

C6.6.8

Load rating of such members should consider second-order effects, which may be approximated by the single-step moment magnification method given in LRFD Article 4.5.3.2.2b (see Appendix C.6.2).

In compression members with asymmetrical sections (such as truss chords), the gravity axis of the section may not coincide with the working lines, resulting in an eccentric connection. Compression members having equal end eccentricities are conveniently analyzed using the secant formula. The LRFD specification does not utilize the secant formula, but provides an interaction equation for the design of members with combined axial loads and concurrent moments. Rating compression members via an interaction equation can be somewhat tedious as an iterative approach may be required to establish the governing rating. A rating approach using the interaction equation is given in Appendix C.6.2. (M_r must be known to apply this method.)

As an alternative to analyzing axial compression members with eccentric connections as combined compression-flexure members, an axial load magnification factor may be applied to rate the member as a concentrically loaded member with an equivalent load. Secant formula is used to include the first and second order bending effects to produce a magnified axial load (dead and live) that would produce a constant stress over the cross section equal to the peak stress in an eccentric member. This approach is applicable to members assumed to be pinned at the ends and without lateral loads on the member. Pin connected compression chord members in truss bridges are a common example of this type. An advantage inherent in this method is that rating factors can be computed without having to first determine M_r, which can be difficult to do for nonstandard truss sections (see Appendix C.6.3).

be checked where the top flange of the girder is fully in contact with the deck.

verify that there is no separation, due to corrosion of the top flange or other causes.

6.6.9.4 Encased I-Sections

Encased I-sections are partially or completely encased in the concrete deck.

If no sign of cracking, rust, or separation along the steel-concrete interface is evident, the encased I-section may be assumed to act as a composite section at the service- and fatigue-limit states. The encased I-section may only be considered composite at the strength limit state if sufficient shear transfer between the steel I-section and the concrete can be verified by calculation.

C6.6.9.4

Encased I-sections constructed without shear connectors may act compositely with the concrete deck due to the bond and friction between the concrete and steel. The degree of composite action varies depending upon the magnitude of loading, degree of encasement of beam flanges, and physical condition of the interface.

Guidance on evaluating composite action in slab-on-girder bridges without mechanical shear connection can be found in NCHRP Research Results Digest, November 1998—Number 234, *Manual for Bridge Rating Through Load Testing.*

6.6.9.5 Moment-Shear Interaction

The shear resistance of stiffened interior web panels subject to the simultaneous action of high moment and shear, as at interior supports of continuous spans, shall be reduced by the moment-shear interaction factor R as specified in LRFD Articles 6.10.7.3.3a and 6.10.7.3.3b. This reduction due to flexure-shear interaction is applicable where $M_u > 0.5\varphi_f M_p$ for compact sections or where $f_u > 0.75\varphi_f F_y$ for non-compact sections.

6.6.9.6 Riveted Members

The moment capacity of riveted sections and sections with holes in the tension flange should be limited to M_y.

C6.6.9.6

At sections of flexural members with holes in the tension flange, it has not been fully documented that complete plastification of the cross section can be achieved prior to fracture of the net section of the flange (see LRFD C6.10.4.1.1).

LRFD criteria could be used for older riveted sections if b/t ratios are satisfied. The Engineer should check the b/t between rivet lines, from the rivet line to the plate edge, and the spacing of the rivets. Net section failure should also be checked. This is dependent upon the yield to tensile ratio of the steel. For riveted compression members, LRFD equations for compressive resistance would be conservative for riveted construction since the riveted members should have much lower residual stresses.

6.6.10 Evaluation for Shear

Shear resistance at the strength limit state is specified in the *AASHTO LRFD Bridge Design Specifications* for I-sections, box-girders, and miscellaneous composite members.

Pin analyses should be performed during the load rating analyses of pin connected bridges because the pins may not necessarily be of equal or greater capacity than the members they adjoin.

The alignment of adjoining members relative to the pin could have a significant effect on the load capacity of the pin as the movement of a member changes the point of application of the member force on the pin. This is especially important on bridges without spacer collars between individual components at a pin. The relative positions of all members that connect to a pin should be ascertained in the field.

The pin size should be measured in the field to ascertain any reduction due to corrosion and wear.

6.6.12.5 Riveted Connections

Riveted connections shall be evaluated as bearing-type connections.

C6.6.12.5

Factored resistance values for rivets are based on AASHTO *Standard Specifications* Article 10.56.1.

6.6.12.5.1 Rivets in Shear

The factored resistance of rivets in shear shall be taken as:

$$\varphi R = \varphi F m A_r \qquad \text{Eq. (6-5)}$$

where:

φF = Factored shear strength of rivet (kips)
m = The number of faying surfaces
A_r = Cross-sectional area of the rivet before driving (sq. in.)

The following values may be used for φF:

Table 6-14 Factored Shear Strength of Rivets φF.

Rivet Type or Year of Construction	φF ksi
Constructed prior to 1936 or of unknown origin	18
Constructed after 1936 but of unknown origin	21
ASTM A 502 Grade I	25
ASTM A 502 Grade II	30

6.6.12.5.2 Rivets in Shear and Tension

Rivets that are required to develop resistance simultaneously to tensile and shear forces resulting from factored loads shall satisfy the following relationship:

$$V_u^2 + 0.56\, T_u^2 \le 0.56\, (\varphi A_r F_u)^2 \qquad \text{Eq. (6-6)}$$

6.7.4.1 Design-Load Rating

Rating factors for the design-load rating shall be based upon the Strength I load combination.

6.7.4.2 Legal load Rating and Permit load Rating

Wood bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

6.7.5 Dynamic Load Allowance

The Dynamic Load Allowance for the evaluation of wood components shall be reduced to 50 percent of the values specified in Articles 6.4.4.3 and 6.4.5.5 for steel and concrete components.

6.7.6 Evaluation of Critical Connections

Critical connections of timber bridges shall be evaluated for shear at the strength limit state.

C6.7.6

External connections of non-redundant members are considered critical connections. Split rings and shear plates may be concealed between wood members. These significantly increase the shear strength of bolted connections. Available records should be consulted to verify their presence. Sometimes a probe may be used to locate them.

6.8 POSTING OF BRIDGES

6.8.1 General

Weight limitations for the posted structure should conform to local regulations or policy, using the guidelines in this Manual. Bridge posting should not be confused with bridge-load rating. Bridge inspection and rating are engineering-related activities, whereas bridge posting is a policy decision made by the Bridge Owner.

Bridges not capable of carrying a minimum gross live-load weight of three tons must be closed. A Bridge Owner may close a structure at any higher posting threshold. When deciding whether to close or post a bridge, the Owner should consider the character of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

A concrete bridge need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. This general rule also applies to bridges for which details of the reinforcement are not known. The bridge should be inspected at regular intervals to verify satisfactory performance.

C6.8.1

Field experience and tests on reinforced concrete bridges (T-beam and slab bridges) have shown that there is considerable reserve capacity beyond the computed value, and that such spans show considerable distress (e.g., cracking, spalling, deflections, etc.) before severe damage and collapse actually occurs.

combination limits and still cause a load effect in excess of that assumed in the rating factor calculation which uses a standard axle distribution. This acute load distribution on the axles has been incorporated in the posting curve.

The reliability level inherent in the posting curve is raised at the lower posting loads to achieve reliability targets closer to design Inventory levels rather than the evaluation or operating reliability characteristic of other practices in this Manual.

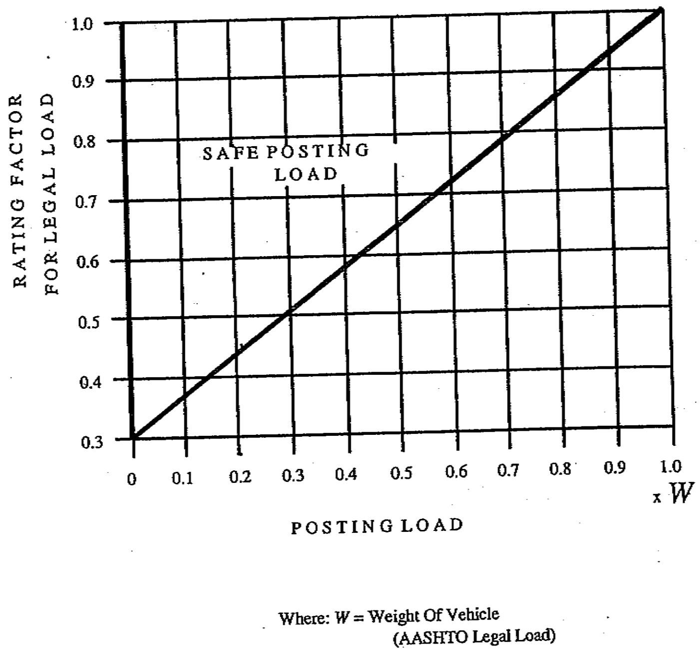


Fig. 6-2 Calculation of Posting Load.

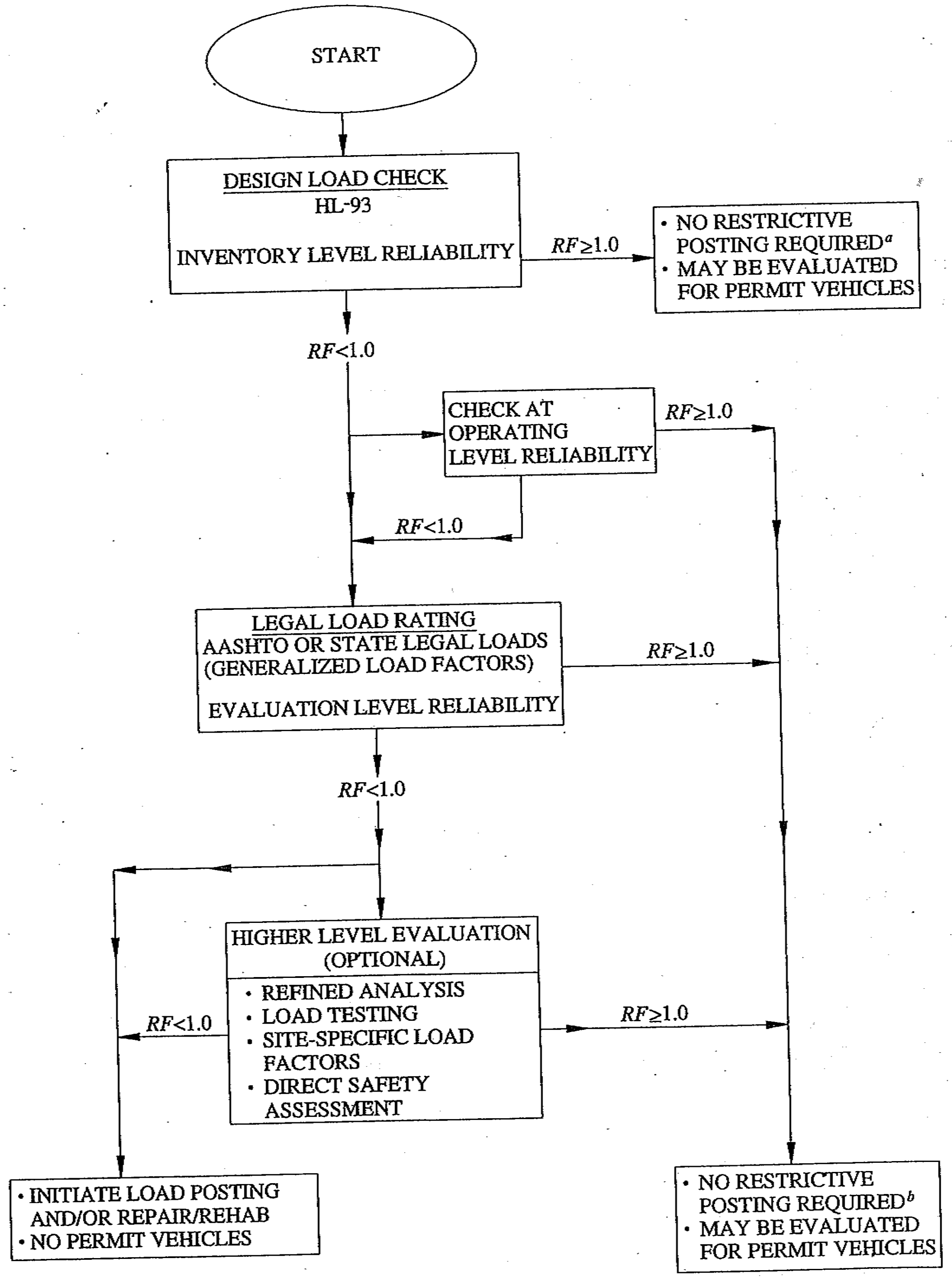
6.8.4 Regulatory Signs

Regulatory signing should conform to the requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD), and should be established in accordance with the requirements of the agency having authority over the highway.

When a decision is made to close a bridge, signs and properly designed, structurally sound traffic barriers should be erected to provide adequate warning and protection to the traveling public. If pedestrian travel across the bridge is also restricted, adequate measures to prevent pedestrian use of the bridge should be installed. Signs and barriers should meet or exceed the requirements of local laws and the applicable sections of the MUTCD. Bridge closure signs and barriers should be inspected periodically to ensure their continued effectiveness.

APPENDIX A.6.1

LOAD AND RESISTANCE FACTOR RATING FLOW CHART



a– For AASHTO legal loads and state legal loads within the LRFD exclusion limits.
b– For AASHTO legal loads and state legal loads having only minor variations from the AASHTO legal loads.

APPENDIX B.6.1

LRFD DESIGN LIVE LOAD (HL-93)
(LRFD 3.6.1)

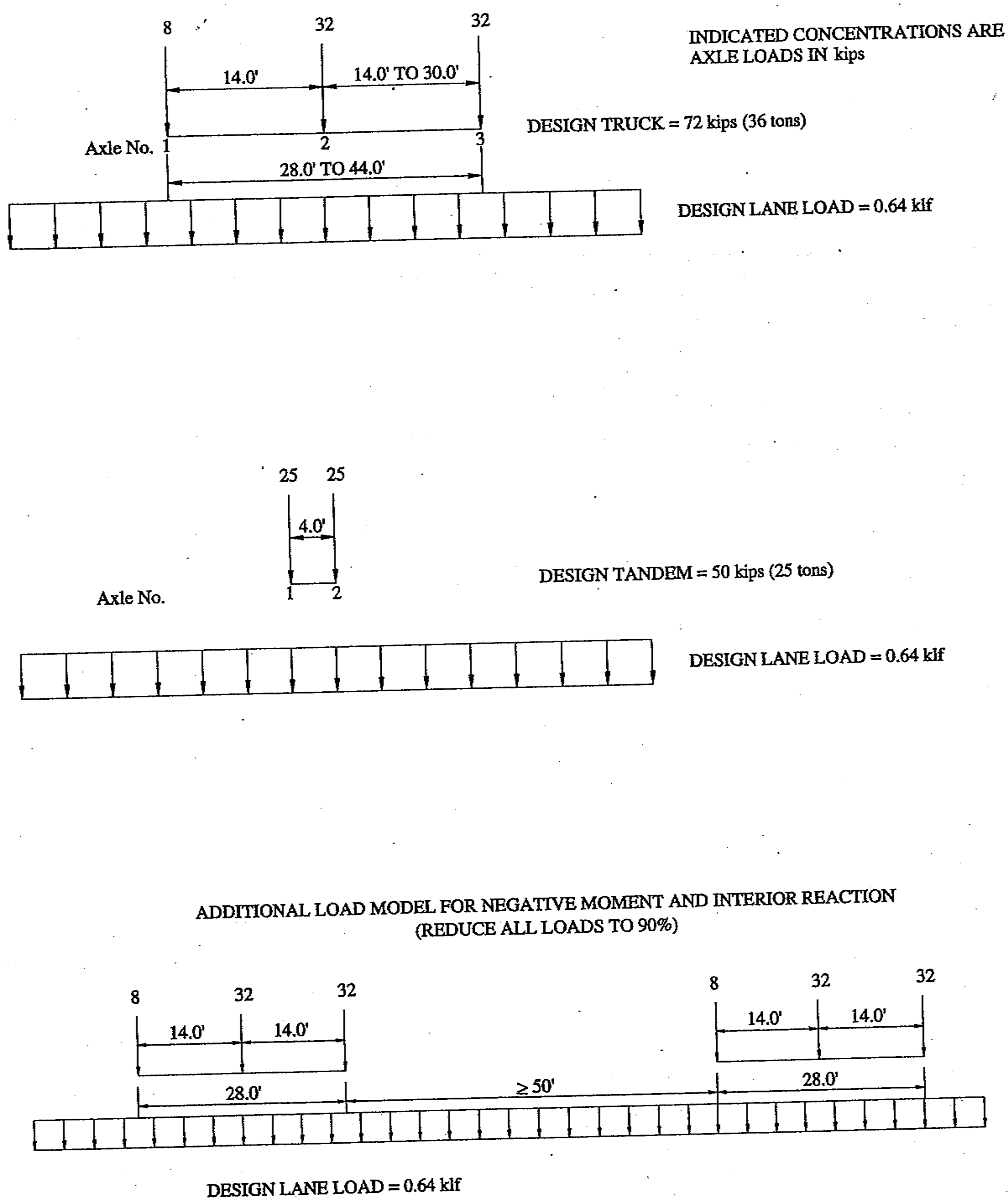


Figure B.6-1 LRFD Design Live Load (HL-93).

b) Lane-Type Legal Load Model—Apply for spans greater than 200 ft. and all load effects.

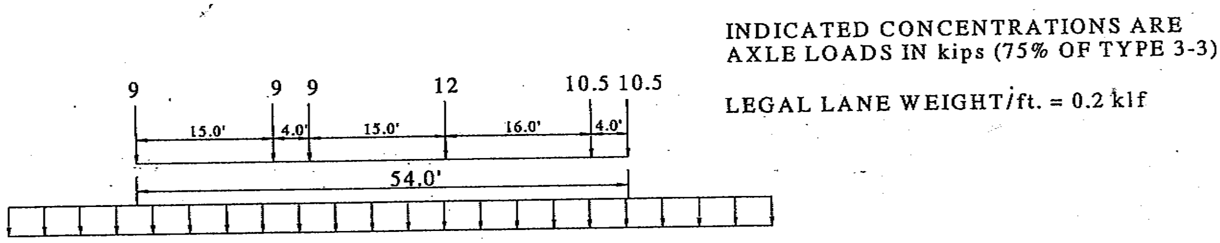


Figure B.6-5 Lane-Type Loading for Spans GreaterThan 200 ft.

c) Lane-Type Legal Load Model—Apply for negative moment and interior reaction for all span lengths.

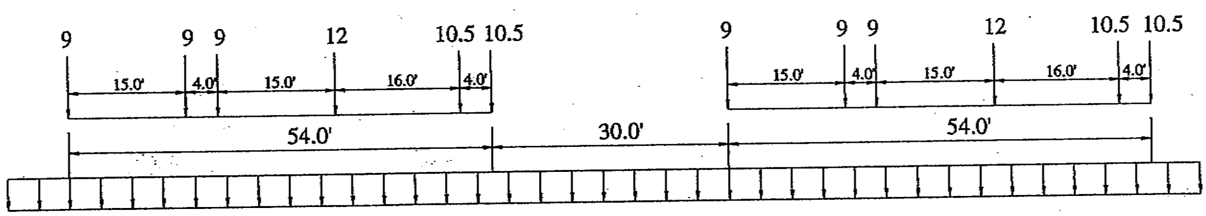


Figure B.6-6 Lane-Type Loading for Negative Moment and Interior Reaction.

APPENDIX B.6.4

VARIATION IN MOMENT RATIO WITH SPAN LENGTH

$$\text{Moment Ratio} = \frac{\text{Simple Span Maximum Moment Caused by the Exclusion Vehicle Population}}{\text{Simple Span Maximum Moment Caused by Each Load Model}}$$

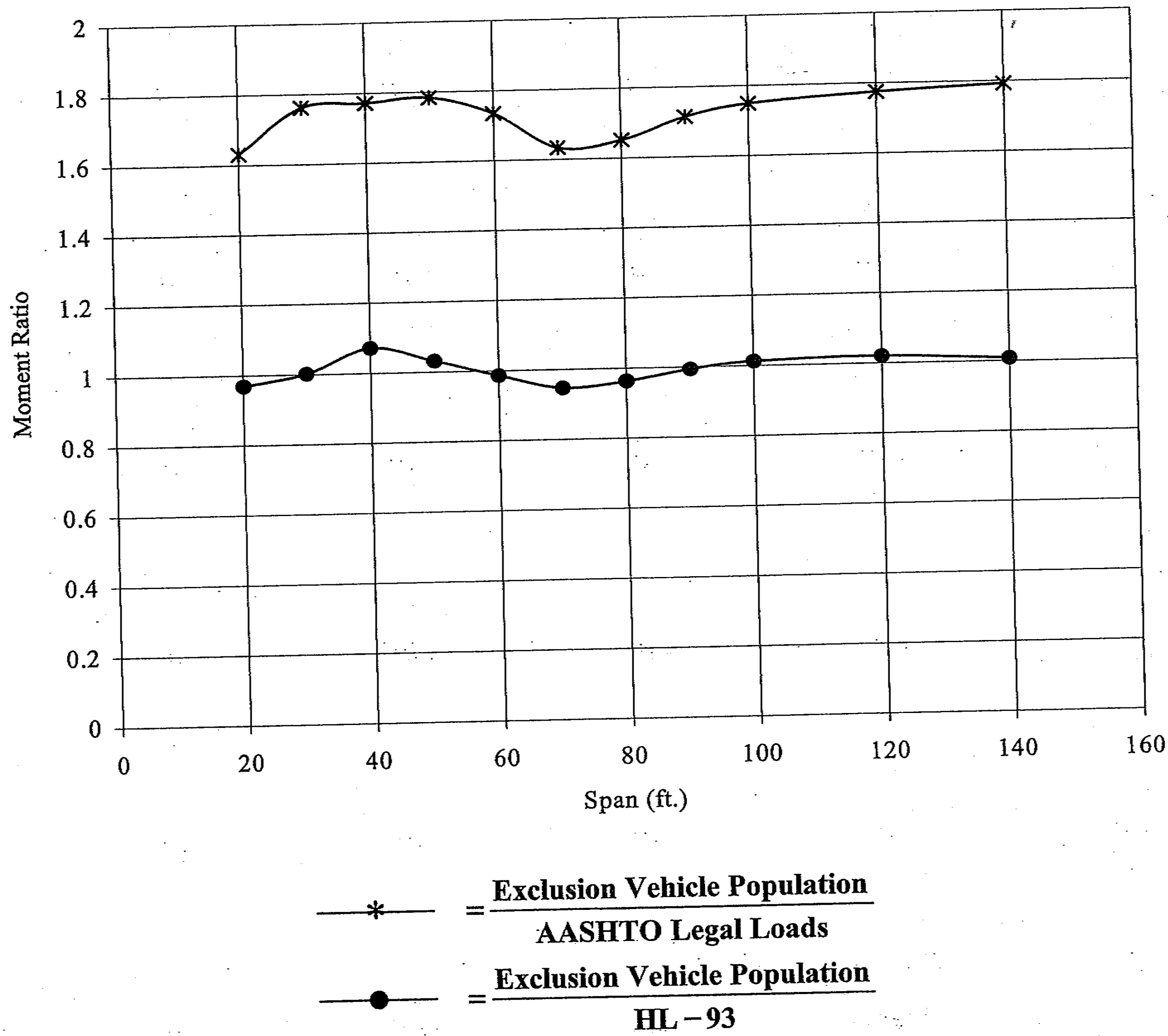


Figure B.6-7 Variation in Moment Ratio with Span Length.

APPENDIX C.6.1

RATING OF CONCRETE COMPONENTS FOR COMPRESSION PLUS BENDING

STEPS FOR OBTAINING RATING FACTORS (see Fig.C.6-1)

1) Develop the interaction diagram, by computer or manual methods, using as-inspected section properties.

2) Point A represents the factored dead load moment and thrust.

3) Using the factored live load moment and thrust for the rating live load, compute the live load eccentricity ($e_1 = M_{LL}/P_{LL}$).

4) Continue from Point A with the live load eccentricity to the intersection with the interaction diagram.

5) Read the ultimate moment and axial capacities from the diagram.

6) Moment RF $= \dfrac{\text{Moment Capacity} - \text{Factored } M_{DL}}{\text{Factored } M_{LL+IM}}$

Axial RF $= \dfrac{\text{Axial Capacity} - \text{Factored } P_{DL}}{\text{Factored } P_{LL+IM}}$

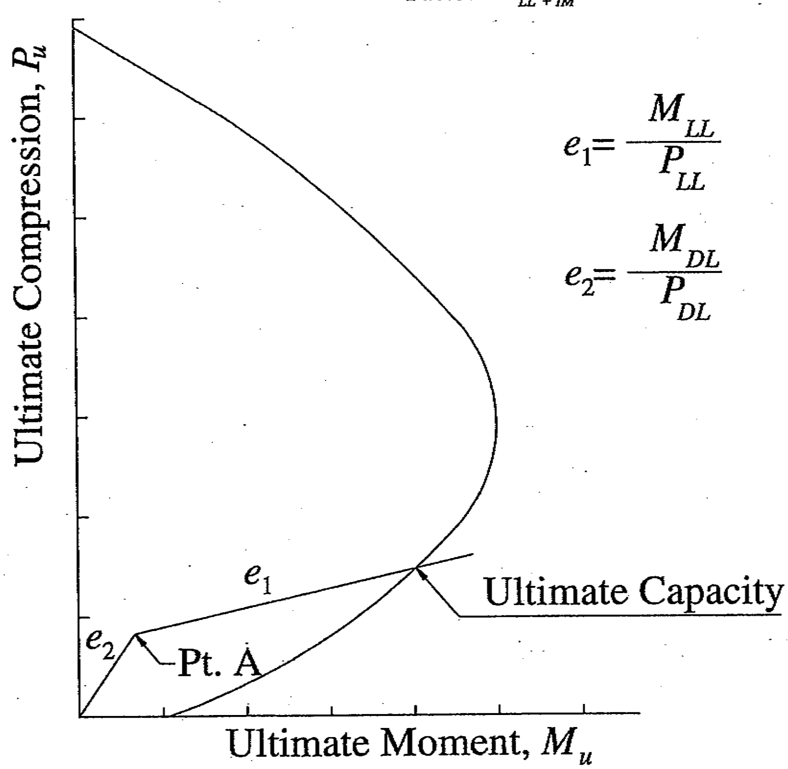


Figure C.6-1 Axial Plus Bending Interaction Diagram for Concrete Structures.

An iterative analysis could be used for improved accuracy.

If, $\frac{P_u}{P_r} < 0.2$

$\frac{P_u}{2P_r} + \frac{M_u}{M_r} \leq 1.0$ for rating the correct RF will make this an equality

LRFD Eq. (6-15)

$$P_u = \gamma_D P_{DL} + (RF)\gamma_L P_{LL+IM}$$

$$M_u = \delta_b \left[\gamma_D M_{DL} + (RF)\,\gamma_L M_{LL+IM}\right]$$

$$\frac{\gamma_D P_{DL} + RF\gamma_L P_{LL+IM}}{2P_r} + \frac{\delta_b \left[\gamma_D M_{DL} + RF \times \gamma_L \times M_{LL+IM}\right]}{M_r} = 1.0$$

$$\gamma_D \left[\frac{1}{2}\frac{P_{DL}}{P_r} + \delta_b \frac{M_{DL}}{M_r}\right] + RF \times \gamma_L \left[\frac{P_{LL+IM}}{2P_r} + \delta_b \frac{M_{LL+IM}}{M_r}\right] = 1.0$$

$$RF = \frac{1 - \gamma_D \left[\frac{P_{DL}}{2P_r} + \delta_b \frac{M_{DL}}{M_r}\right]}{\gamma_L \left[\frac{P_{LL+IM}}{2P_r} + \delta_b \frac{M_{LL+IM}}{M_r}\right]}$$

Where: (for RF = 1.0)

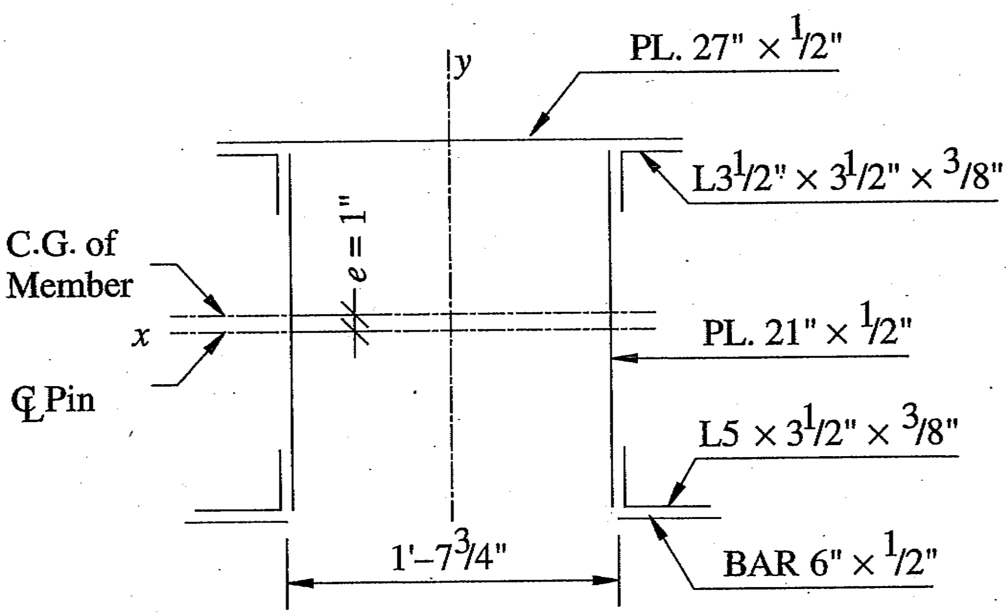
$$\delta_b = \frac{C_m}{1 - \frac{\gamma_D P_{DL} + \gamma_L P_{LL+IM}}{\varphi\, P_e}}$$

An iterative analysis could be used for improved accuracy.

Example rating using axial load magnification:

Section based upon member in Appendix A Example 6 but with the pins assumed to be 1 in eccentric in the negative y coordinate. Member forces calculated assuming centerline of pin to be concentric with center of gravity of top chord.

e	= 1 in.	A	= 55.3 in.^{2}	S_x *bottom*	= 376.0 in.^{3}
L	= 300 in.	E	= 29000 ksi	I_x	= 5716.8 in.^{4}
P_{DC}	= 558.1 kip	P_{DW}	= 39.4 kip	P_{LL+IM}	= 231.1 kip



$$P_u = 1.75\times 231.1 + 1.25\times 558.1 + 1.25\times 39.4 = 1151.3 \text{ kip}$$

$$\delta_A = \left(1+\frac{1\times 55.3}{376.0}\sec\left(\frac{300}{2}\sqrt{\frac{1151.3}{29000\times 5716.8}}\right)\right)$$

$$\delta_A = 1.159$$

$$RF = \frac{0.85\times 1.0\times 0.9\times 1906.6 - 1.25\times 1.159\times 558.1 - 1.25\times 1.159\times 39.4}{1.75\times 1.159\times 231.1}$$

$$RF = 1.26$$

(RF = 1.76 for e = 0 in Example 6.)